Vol. L. MAY, 1924. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

INCREASING THE CAPACITY OF EXISTING STREETS*

By Arthur S. Tuttle,† M. Am. Soc. C. E.

It is only a few years since the public depended wholly on the horse-drawn vehicle for free-moving transportation. The introduction and popularizing of the motor car has created a problem which the authorities in nearly every city are endeavoring to solve in order that street capacity may be made adequate for the ever-increasing demands of traffic.

City planning in New York began in 1811, when a commission appointed by Mayor DeWitt Clinton prepared a map for that part of the old city—now the Borough of Manhattan—extending from the developed area south of Houston Street and Greenwich Village northerly to West 155th Street, a distance of about 7 miles. This map made provision for a series of parallel north and south avenues, each 100 ft. wide, spaced 610 to 920 ft. apart, and at right angles a series of parallel east and west streets 200 ft. apart, with widths of 60 ft., except at intervals of seven to fifteen blocks where widths of 100 ft. were established. This plan, later extended northward to the Harlem River, largely constitutes the Manhattan Plan of to-day. Until recently, it was considered to have provided more than an abundance of street space. As a result, invasions have been permitted to such an extent as seriously to impair the usefulness of a plan which, in breadth of vision, compares favorably with present-day planning in any of the large cities. It would seem that, although the city inherited a layout of great merit, the inheritance was not valued sufficiently by the citizens of later years, otherwise, it would be much easier to remove the various impediments to traffic that have been introduced.

Present conditions in New York are probably typical of those of other large cities, but more acute. Statistics on the traffic across the East River bridges indicate an average annual increase of about 20% for the last 10 years, and the Police Commissioner states that the number of vehicles in the city is increasing at the rate of about 30% annually. From a recent study of all available data, it has been estimated by the writer that 15% represents a conservative estimate of the present annual increase in traffic in the congested districts, with the probability that the rate would be much higher if the street capacity was more adequate.

Police regulations have been carried as far as seems practicable under present conditions. One-way east and west traffic is in force through the congested districts and the two main north and south arteries are controlled by signal systems. It is acknowledged that further relief is urgently needed in order to cope with the present, to say nothing of the constantly increasing, traffic needs.

---

* Presented at the meeting of the City Planning Division of the Society, January 17, 1924. Written discussion on this paper will be closed with the September, 1924, *Proceedings*. When finally closed, the paper with discussion will be published in *Transactions*.

† Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

The widening of streets after they have been fully improved, or the breaking through of new avenues heretofore observed is expensive, usually involves the complete destruction of the buildings affected as well as heavy consequential damages to land, and frequently leaves the frontage so badly gored as to be unsuitable for further substantial use.

It has been estimated that to cut a new express street 100 ft. wide through the heart of the Borough of Manhattan would cost about $25 000 000 per mile. Excess condemnation, for the purpose of salvaging remnants, would temporarily add still greater burdens and would involve so much capital that such a project in this part of the city would be largely impracticable.

In order to provide the relief now demanded without embarking on a plan involving the utilization of unduly large areas devoted to private use and without destroying valuable buildings, a number of expedients have been, or are being, considered, as follows:

1.—Requiring new buildings to be set back to lines proposed as the ultimate limits of the widened street.
2.—Widening of roadways.
3.—More drastic traffic regulations.
4.—Removal of roadway encroachments.
5.—By-passing of traffic.
6.—Introduction of arcades.

1.—Set-Back Ordinance.—In some cities, street widenings have been accomplished through legislation prohibiting the erection of a building other than in conformity with the lines for a new street or for the widening of an existing street, thereby limiting the cost of widening a street which had been fully built up to the value of the land, provided that the proceeding was deferred until after all the old buildings had been replaced. In the meantime, it seems probable that the land needed for street purposes would have become burdened with public easements to such an extent as to have only a nominal value. Such a practice was observed in New York until 1893, when its legality was contested in the Courts. The opinion rendered in this case was as follows:

"Whenever a law deprives the owner of the beneficial use and free enjoyment of his property, or imposes restraints upon such use and enjoyment that materially affects its value, without legal process or compensation, it deprives him of his property within the meaning of the Constitution, and, although the police and other powers of government may sometimes incidentally affect property rights, these powers can only be exercised to promote the public good, and are always subject to judicial scrutiny.

"The provision of the New York Consolidation Act (Section 677, Chapter 410, Laws of 1882), which declares that no compensation shall be allowed to the owner of land taken for a street for any building erected or placed thereon after the filing of a map of the street as prescribed by the Act (Section 672) by its terms, imposes a restriction upon the use of the land, which amounts to an encumbrance, and so is unconstitutional."

The unconstitutionality of such a law seems clear both on the basis of the Federal as well as of the State Constitutions, and the decision has since been accepted in New York. Efforts elsewhere to break down such a law

will be made when the land and building values are sufficiently high to warrant litigation and must inevitably meet with success.

In this connection, it might be noted that, under the Zoning Act, the city authorities of New York have recently imposed a 15-ft. set-back on the highest class of residential property and now have under consideration the establishment of a 10-ft. set-back for residential areas where the restrictive conditions are less severe. These restrictions are being enforced under the police powers of the State and are designed to protect a particular type of development to which an area seems best adapted. It does not seem probable that any such requirement could be successfully imposed where development had been well advanced and was primarily of another type, or that similar means of relief could apply to congested sections.

2.—Widening of Roadways.—During 1908, the congestion of traffic on Fifth Avenue, which is the only north and south street in Lower Manhattan free from obstruction, became so acute that the roadway between 25th Street and 47th Street was widened from 40 ft. to 55 ft., the sidewalk widths being decreased from 30 ft. to 22.5 ft. At that time, Fifth Avenue was a residential street. This improvement involved the removal of entrances, steps, and fences. The reconstruction which at once took place converted it into a high-class business thoroughfare and was followed by an extension of the roadway widening to include the entire length of the street south of Central Park. This widening marked the first important move toward the freeing of the streets as far as possible to meet the needs of vehicular traffic without increasing the street width.

A large number of similar improvements in the more congested parts of the Borough of Manhattan have since been made, particularly in streets having a width of 60 ft., with 30-ft. roadways and 15-ft. sidewalks, originally developed for residential use. A substantial part of the sidewalk area has been appropriated in a manner similar to that already described in Fifth Avenue. The width of these roadways is being increased to 34 ft. with repaving improvements and, through the removal of encroachments, practically all the space available to pedestrians is being conserved. This has caused a marked relief in districts having light traffic, as it permits two lanes of vehicles moving in opposite directions and a lane of traffic parked at each curb. In districts where the streets are used by large trucks, it is evident that traffic must still be seriously impeded.

The expediency of still further decreasing the sidewalk allowance, which now generally averages from 13 ft. to 20 ft. in width, has recently been questioned, but it is evident that the resulting gain would not accomplish much relief.

In the planning of vehicular tunnels where construction costs are high, careful consideration has been given to the kind of traffic and the space required for its accommodation. It does not appear that heretofore any such serious consideration has been given to the planning of a street, but by reason of the great expense involved in the congested areas, it seems evident that this phase of the problem should be considered. The number of lanes of traffic must be determined and space must be provided for each.

Traffic regulation requirements seem to call for a permanent demarcation between lanes used for movement in opposite directions, and it may be expected that eventually each lane will be clearly marked on the roadways. This being true, it follows that street widths should be fixed in units which will accommodate two lines of parked traffic and an even number of moving lines, and should not be increased at random in small units as heretofore has generally been done.

3.—Traffic Regulation.—The introduction of traffic regulation at 42d Street and Fifth Avenue during 1903 has been followed not only by its extension to all busy intersections throughout the city, but also by imposing one-way traffic rules on the narrower streets where there is congestion and by establishing traffic-tower control over some of the most important arteries. These methods of handling the traffic problem have been improved to a point of efficiency which is believed to be unsurpassed. The Police Commissioner has recently announced, however, that regulation under present conditions has reached its limit and that relief must come through the introduction of new devices.

Pending the provision for more street area, it would seem that a substantial gain in capacity might be obtained through the limitation of arterial streets to one-way traffic and the introduction of the block system of control, the latter being dependent on the former. One-way traffic regulation requires a re-routing of surface cars and bus lines so that all movement will flow in one direction; it can only be effected through co-operation between the municipal authorities and the operating companies. Its adoption would doubtless meet with opposition from the business interests affected which, however, are bound to suffer through the perpetuation of present conditions, to say nothing of the certainty of continuously increasing congestion unless relief is found. The block system of operation resembles that used for railroads and, if properly controlled, should permit traffic in the controlled area to proceed continuously through it without halting, the section thus controlled becoming a semi-expressway. This arrangement, with due allowance for increasing the distance between cars as required for safety, should have the effect of speeding up the movement at least 25%, and thus afford a substantial increase in street capacity.

4.—Removal of Roadway Encroachments.—Until recently, roadways have been regarded as having surplus width and their invasion, beginning with surface car tracks and steam railroads, has extended to elevated railroad columns, subway station entrances, subway ventilator gratings, and malls. These utilities have served in the growth of the city and when the municipal authorities announced about four years ago that the day for further extension of elevated railroads in the city had passed, the statement was at first not taken seriously. However, not only is this policy now generally accepted, but active efforts are being made to restore the streets to the use for which they were intended.

The seriousness of the situation is strikingly illustrated by recent traffic counts of the north and south movement across 42d Street at all the arteries between the Hudson and East Rivers. From these studies it was found that the movement ranges from a headway of about 5 sec. in unobstructed Fifth

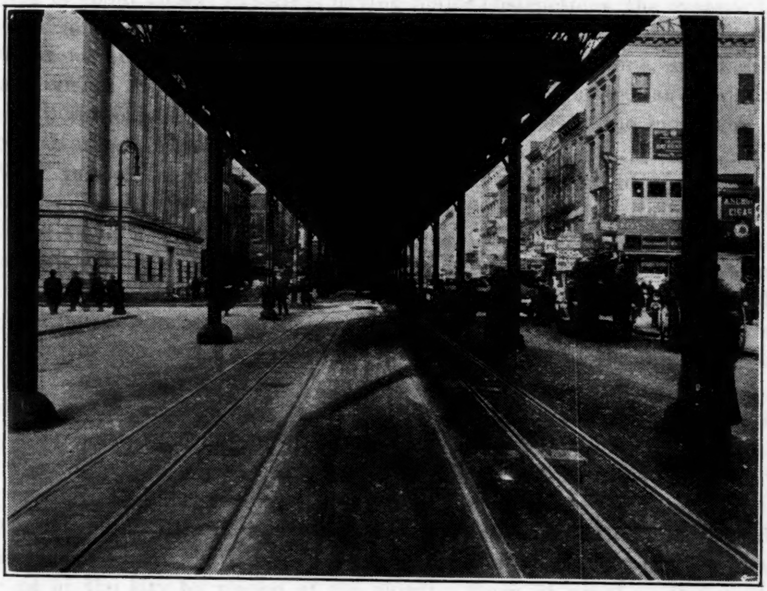


FIG. 1.—VIEW OF SIXTH AVENUE, NEW YORK, N. Y., LOOKING NORTH BETWEEN 35TH AND 36TH STREETS.

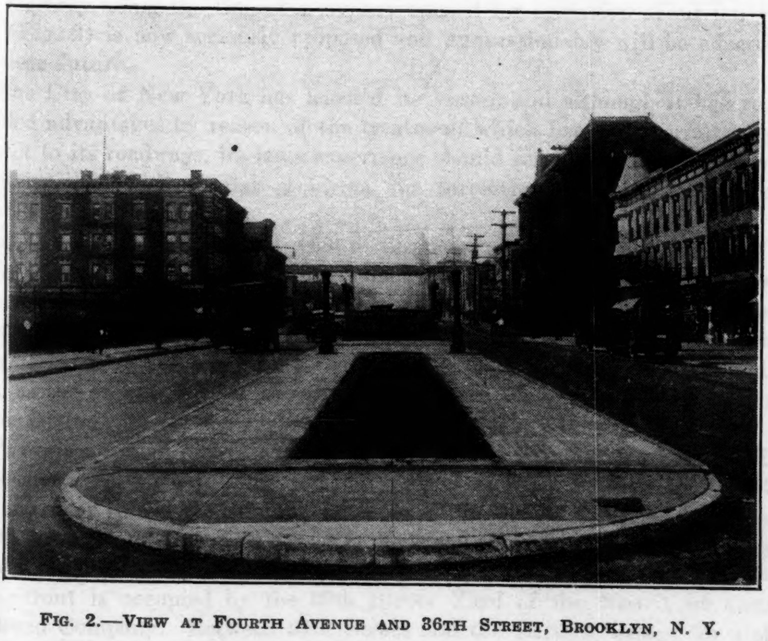


FIG. 2.—VIEW AT FOURTH AVENUE AND 36TH STREET, BROOKLYN, N. Y.

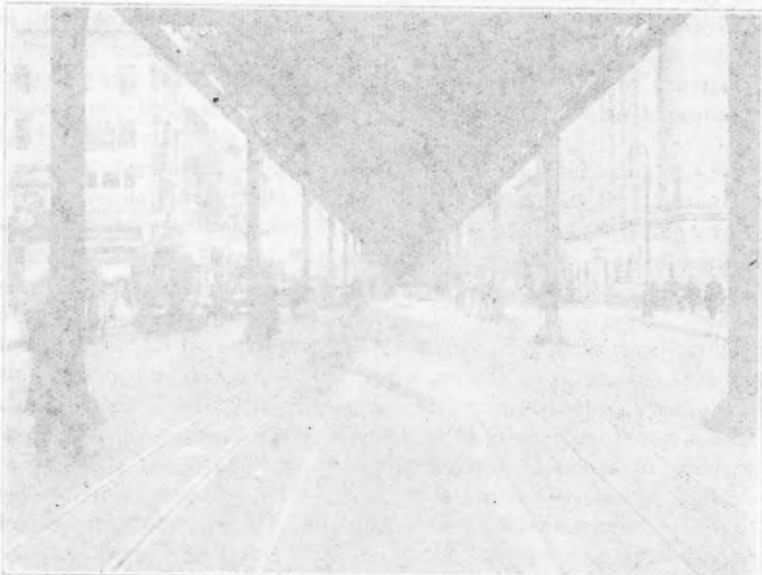


FIG. 1.—VIEW OF SIXTH AVENUE, NEW YORK, N.Y., LOOKING NORTH BETWEEN 40TH AND 41ST STREETS.

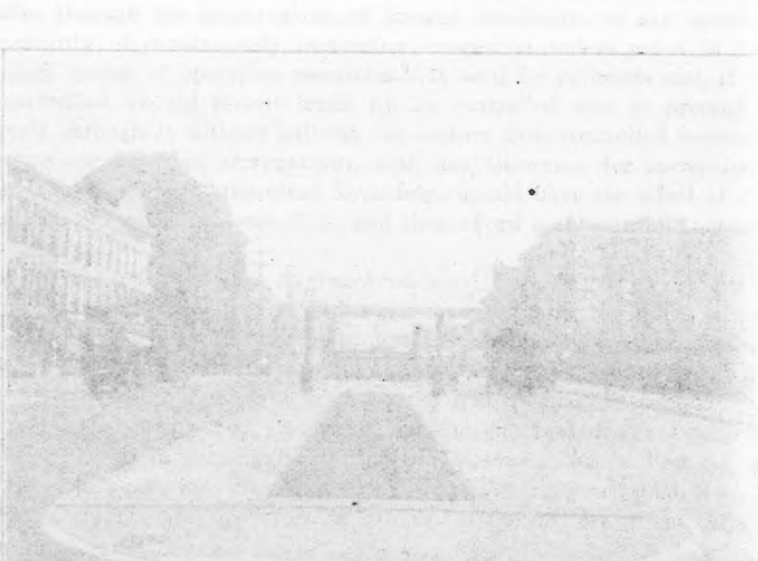


FIG. 2.—VIEW AT FOURTH AVENUE AND 86TH STREET, BROOKLYN, N.Y.

Avenue to about 25 sec. in streets having major obstructions, the average headway for all being about 9 sec. It would appear, therefore, that if all the obstructions, such as surface tracks, elevated railroad columns, and the West Side Division of the New York Central, which is operated at grade, were removed, the capacity of the north and south streets in the most congested part of Manhattan could be increased about 80%.

With this object in view, the President of the Borough of Manhattan has recently proposed the removal of the Sixth Avenue Elevated Railroad and the substitution of a subway, thereby freeing the street, except for surface cars, to vehicular use and restoring the light and air of which it is now to some extent deprived. (Fig. 1.) The adjacent property owners have shown a keen appreciation of the benefits that would follow such an improvement, even to the extent of expressing a willingness to bear the cost of the improvement in the belief that the benefits would more than equal the expense.

The surface tracks of the New York Central Railroad have constituted a serious problem, the solution of which cannot long be deferred in the interest of either the public, whose safety and convenience are seriously prejudiced, or the corporation, which is subjected to great difficulties in operation.

The economic advantages of the surface car over the bus and taxicab in street capacity used, have been to a considerable extent lost in the crowded sections of the city by reason of the slowing down of speed as the result of the absence of freedom of movements, which slowing down is also shared with all the traffic using the same street.

The removal of subway malls and station entrances from the middle of the roadway along the line of a subway opened for operation about ten years ago (Fig. 2) is now seriously proposed and unquestionably will be effected in the near future.

The City of New York has learned its lesson, and although it has reaped decided advantages by reason of the treatment which has been permitted with respect to its roadways, its later experience should serve as a warning to other municipalities with similar problems, the correction of which is certain to be both costly and slow.

5.—By-Passing of Traffic.—One of the most promising fields for relief lies in the diversion of through traffic from congested sections into streets giving greater freedom of movement or more economical increase of highway capacity. This remedy is being applied in New York as far as possible in the planning of boulevards in the outlying districts, but is only available to a limited extent, as the greater part of the traffic that enters the city is seeking the areas of congestion.

A project for an expressway along the Hudson River water-front has been suggested from time to time for many years and has recently been revived by the writer in a more comprehensive way. Traffic on Riverside Drive south of its terminus at West 72d Street is dispersed through various arteries, none of which stands out pre-eminently. Between 72d Street and 59th Street, the water-front is occupied by the 60th Street Yard of the New York Central Railroad Company. Between 59th Street and the Battery, except for a short distance south of 42d Street, the city has acquired a strip of land about 150 ft.

wide for a marginal way to serve the water-front. This strip in turn is flanked by West Street, with a width of about 100 ft., designed for the accommodation of the vast trucking business incidental to the service of the piers. The marginal way is mostly paved, and this, as well as West Street, is usually congested with a tangled mass of vehicles awaiting an opportunity to discharge or receive the pier freight. It is proposed to construct a viaduct over the 60th Street railroad yard and a shed over the marginal way to a depth of about 75 ft. Over this it is proposed to construct an expressway 100 ft. wide with a 25-ft. overhang toward West Street. This arrangement would make it possible to distribute pier freight from a landing platform extending practically the entire length of West Street, against which, and in the covered area outside the street, the trucks could back up for loading or unloading, thereby avoiding the present confusion, clearing West Street, and conserving time. It would seem that such a project would greatly decrease the present cost of handling freight at the water-front, and, incidentally, make it possible to recoup a considerable part of the cost of the express street. Such a street would be provided with ramps to grade at intervals of about a mile. With its ten moving traffic lanes, it would provide a capacity more than twice as great as that of Fifth Avenue; assuming a conservative increase in speed, it would seem that the new street would accommodate three times as many vehicles as Fifth Avenue.

In the development of such a project, it would also seem feasible to move the express street up to a third level and to introduce in the middle level a marginal water-front railroad which could easily be connected with warehouses on the easterly side of West Street and provided with facilities for directly receiving or discharging marine or local freight (Fig. 3).

6.—Arcaded Sidewalks.—In order to provide more space, it would be practicable to double-deck streets, but the introduction of such elevated structures, except along the water-front, is not regarded by the writer with favor, his objections being similar to those against elevated railroads. Furthermore, double-decking would depreciate property values on the lower level even more than elevated structures, and the construction would involve costly building and consequential damage in ramping to grade.

There lies an intermediate procedure between this course and an actual street widening, that is, to provide a sidewalk in an arcade back of the street line and wholly within the fronting buildings.

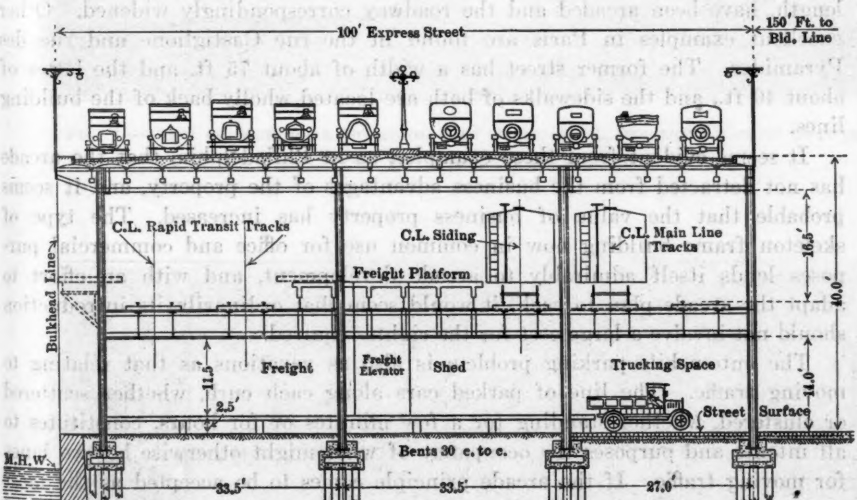
The only known example of such an arrangement in the United States for the purpose of devoting to roadway use what would ordinarily be a sidewalk, is noted* by the late Nelson P. Lewis, M. Am. Soc. C. E., as follows:

"Philadelphia again furnishes a notable example. Fifteenth Street, between Market Street and South Penn Square, has a width of 50 ft., with a roadway of 26 ft. and sidewalks of 12 ft. each. The curb on the east side was set back 11 ft., or within 1 ft. of the street line, while a sidewalk 19 ft. in width was provided back of the new curb line extending 18 ft. under the buildings. This improvement was also extended eastward along the north side of South Penn Square between 15th and Broad Streets. The arcade thus formed has been

* Paper presented at the National Conference on City Planning, 1917.

treated uniformly, the supporting piers are regularly spaced and all of the same size, and the results of this treatment have been very satisfactory, the shops fronting on the arcade appearing to be desirable and probably commanding good rentals. The total length of this arcade on both streets is 335 ft., and the cost to the City in damages paid for the easements and the reduction of available floor space in the buildings amounted to $193 000. Consideration is now being given to the extension of this arcade along the east side of 15th Street to Chestnut Street, a distance of about 200 ft. It should be pointed out, however, that such treatment cannot be successful unless the arcades are of uniform height and unless the supporting columns and the façades of the buildings themselves are harmoniously designed."

In order to secure the desired floor space, the Municipal Building in New York was arched over Chambers Street, through the block between Park Row and Center Street, the sidewalks being placed in an arcade (Fig. 4). In this instance, however, there was no occasion for equipping the arcade with show windows such as would ordinarily be required, and there was no damage to abutting property. A somewhat similar treatment is proposed for the building to be erected by the New York Central Railroad Company in Park Avenue, between 45th Street and 46th Street, as a part of the extensive improvement involving a high-level street along the easterly side of the Grand Central Terminal.



SUGGESTED EXPRESS STREET, MARGINAL RAILROAD AND FREIGHT SHED TO BE LOCATED ON THE MARGINAL WAY ALONG THE HUDSON RIVER SOUTH OF 59th STREET

FIG. 3.

Possibly, the practical advantages of the arcade are better exemplified by the building now being erected by the New York Telephone Company, extending along the northerly side of Vesey Street, from West Street to Washington Street. Vesey Street here has a width of about 56 ft., and it was proposed by the City authorities to take advantage of the opportunity which

would be afforded by the removal of the old buildings to increase the street width to about 80 ft. The Telephone Company considered the project to be objectionable on account of the restricted space which would result for building purposes and, as a compromise, proposed that if the street widening was abandoned, it would give the city sidewalk privileges within the building to a width of about 20 ft., as shown in Fig. 5. Hitherto, the street arcade had been regarded as somewhat impracticable because of what appeared to be a serious difficulty in acquiring, under formal proceedings, a form of title that would enable the city authorities to construct a sidewalk without at the same time depriving the owner of rights that might be essential to a convenient or proper use of his property. The studies made in this instance disclosed the practicability of effecting an adjustment fair to both parties, and of establishing the legality of the procedure. It should be noted, however, that any such proceeding conducted on a scale which affected several properties would have to be developed along lines adapted to each individual parcel.

In Southern Europe, the arcaded sidewalk seems to be quite common, particularly in cities where the climatic conditions are such as to make its use advantageous as a protection from severe snowstorms. The most notable example is the rue de Rivoli in Paris; this street has a width of about 70 ft. and the buildings on the northerly side, through a considerable part of its length, have been arcaded and the roadway correspondingly widened. Other excellent examples in Paris are found in the rue Castiglione and rue des Pyramides. The former street has a width of about 75 ft. and the latter of about 40 ft., and the sidewalks of both are located wholly back of the building lines.

It seems evident from these examples, as in Philadelphia, that the arcade has not detracted from the business advantages of the property, and it seems probable that the value of business property has increased. The type of skeleton frame building now in common use for office and commercial purposes lends itself admirably to arcade development, and with an effort to adapt the arcade plan to each, it would seem that ordinarily its introduction should not involve a large cost for the rights required.

The automobile parking problem is also as vexatious as that relating to moving traffic. The line of parked cars along each curb, whether scattered or clustered, whether standing for a few minutes or for hours, constitutes to all intents and purposes the occupancy of what might otherwise be two lanes for moving traffic. If the arcade principle comes to be accepted as the most practicable remedy for congestion where property values are high, it would seem logical to extend it in such a way as to place the sidewalk on the second level, with bridges across intersected streets, and to devote the lower arcade level to parking use, thus not only increasing the roadway space, but also separating the grades for pedestrian and vehicular traffic, thus greatly increasing the safety of walkers and speeding up vehicles.

In a recent study of the advantages of the arcade treatment for the north and south avenues of New York, the conclusion was reached that the assignment of sidewalk space to roadway use would nearly double the number of available traffic lanes, and that from every point of view this method was the

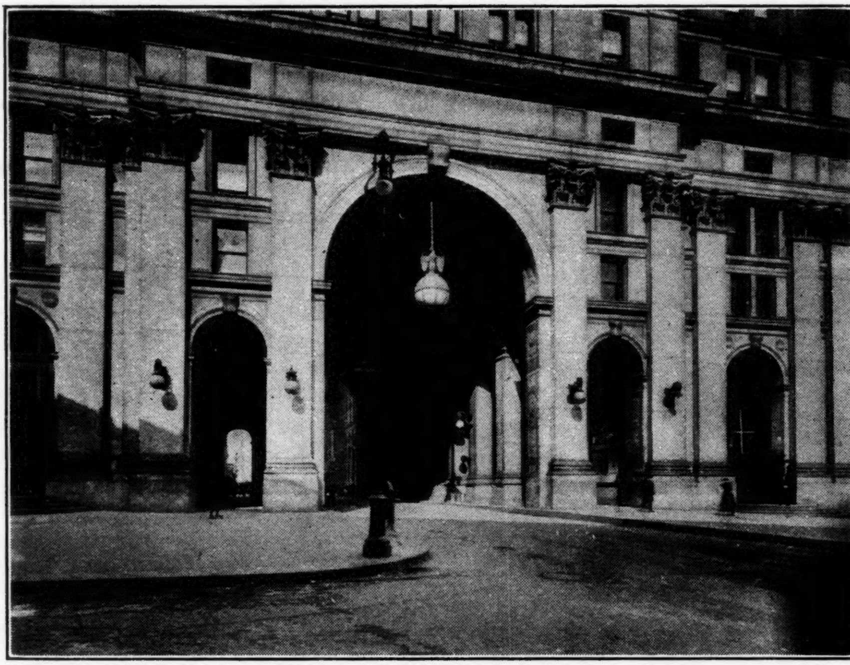


FIG. 4.—VIEW OF MUNICIPAL BUILDING, NEW YORK, N. Y., SHOWING ARCADE BETWEEN PARK ROW AND CENTER STREET.

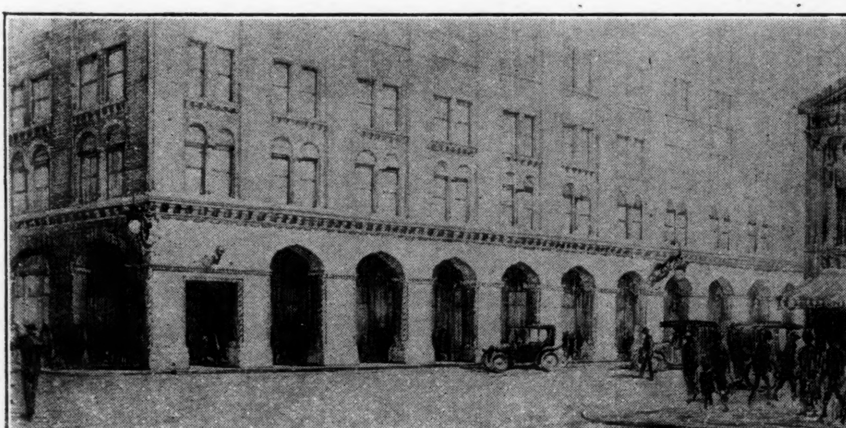


FIG. 5.—VIEW OF PROPOSED ARCADE ON VESEY STREET, NEW YORK, N. Y.

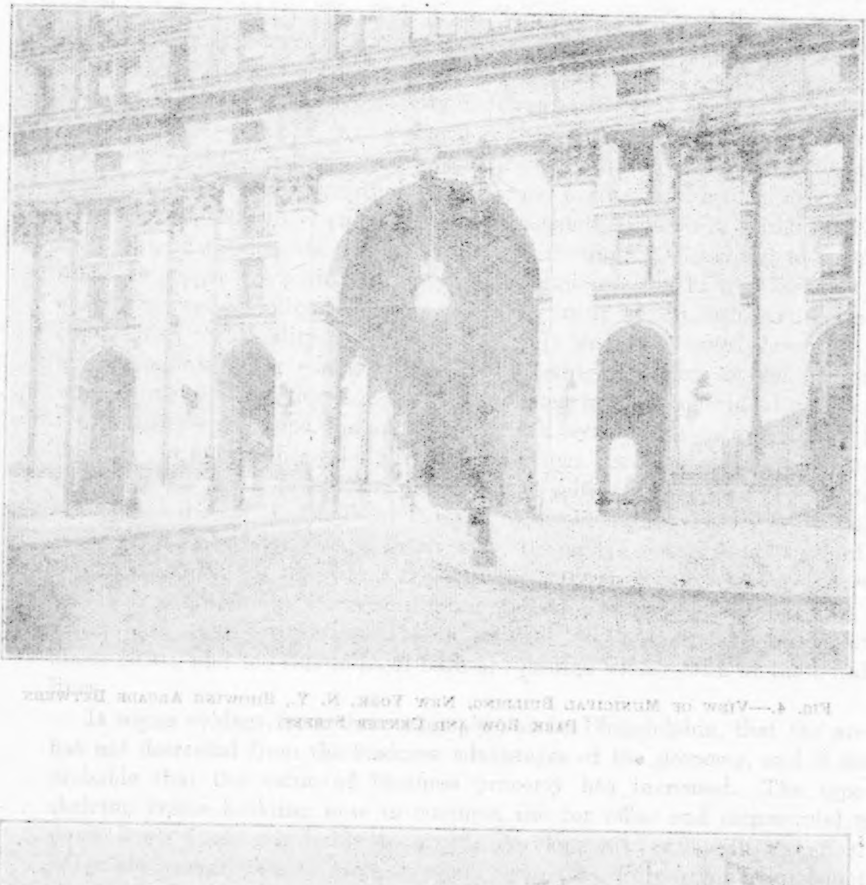


FIG. 4.—VIEW OF MUNICIPAL BUILDING, NEW YORK, N. Y., SHOWING ARCADE BETWEEN PARK ROW AND CENTER STREET.

FIG. 5.—VIEW OF PROPOSED ARCADE ON VESEY STREET, NEW YORK, N. Y.

most practicable for the economical and comprehensive remedying of congestion.

Heretofore, the traffic relief problem has been regarded as one in which the Police Department was primarily concerned, but it would seem that it has now assumed such proportions that the engineering service must take the lead in providing a solution. The recognition of this responsibility by the profession should bring about the freeing of the streets for the traffic needs of to-day and the days to come.

EXPLORATIONS IN THE DESERT REGION OF CENTRAL ASIA*

By Charles P. Berkey,† Esq.

Two years ago, about this time (January, 1922), the speaker sailed across the Pacific to join the Third Asiatic Expedition, which was then planning explorations in Central Asia. Considering their scientific possibilities, the desert regions of Central Asia are the least known of any traveled regions of the earth. Judging from many indications, Asia has been a continent of large extent and favorable conditions for the development of animal life for so many geological ages that it should prove to be a particularly promising ground for investigations, not only into the geologic history of the continent in general, but also into the succession of different forms of animal life. Long ago, explorations in America unearthed many rare and interesting forms, the relations and genetic connections of which were not by any means fully represented. Breaks in the series seemed to point to the possibility of parallel development of similar forms in other regions, at times quite independent of those in North America, and, perhaps, at other times connected with them. To those acquainted with the geological evidence, it appeared reasonable to believe that there was, in different epochs, a land connection between North America and some other continent, probably across the Bering Sea region from America to Asia.

In view, therefore, of the peculiarly favorable conditions that were believed to characterize the very ancient Asiatic Continent, it was perfectly natural to look to Central Asia as a likely field in which to find the complementary forms and events belonging to a more complete geologic and life story. Ancient animals of many kinds should be found there, and man, also, if perchance he lived with them. In any case, the geologic data to be gathered must throw much light on the geologic history of Asia.

Observations and evidence that may lead to a better understanding of the problems of the earth—these are the logical objects of any scientific expedition. This is what is meant, in a broad way, by looking for the "missing link". There are in all such unexplored regions many kinds of missing links, and to the geologist, the paleontologist, and the geographer, the task is to search them out, determining what they are, and find their meaning. Thus, may one slowly arrive at a better explanation of the way things have come to be as they are. This, in fact, is what the Third Asiatic Expedition set out to do.

The region chosen was Central Mongolia. There it was found possible to plan a traverse aggregating more than 3 000 miles into the heart of Asia, almost all of it virgin territory. (Fig. 1.)

* Abstract of an illustrated address presented at the Annual Meeting of the Society, January 17, 1924, published by permission of the American Museum of Natural History.

† Prof. of Geology, Columbia Univ.; Chf. Geologist, Third Asiatic Expedition, American Museum of Natural History, New York, N. Y.

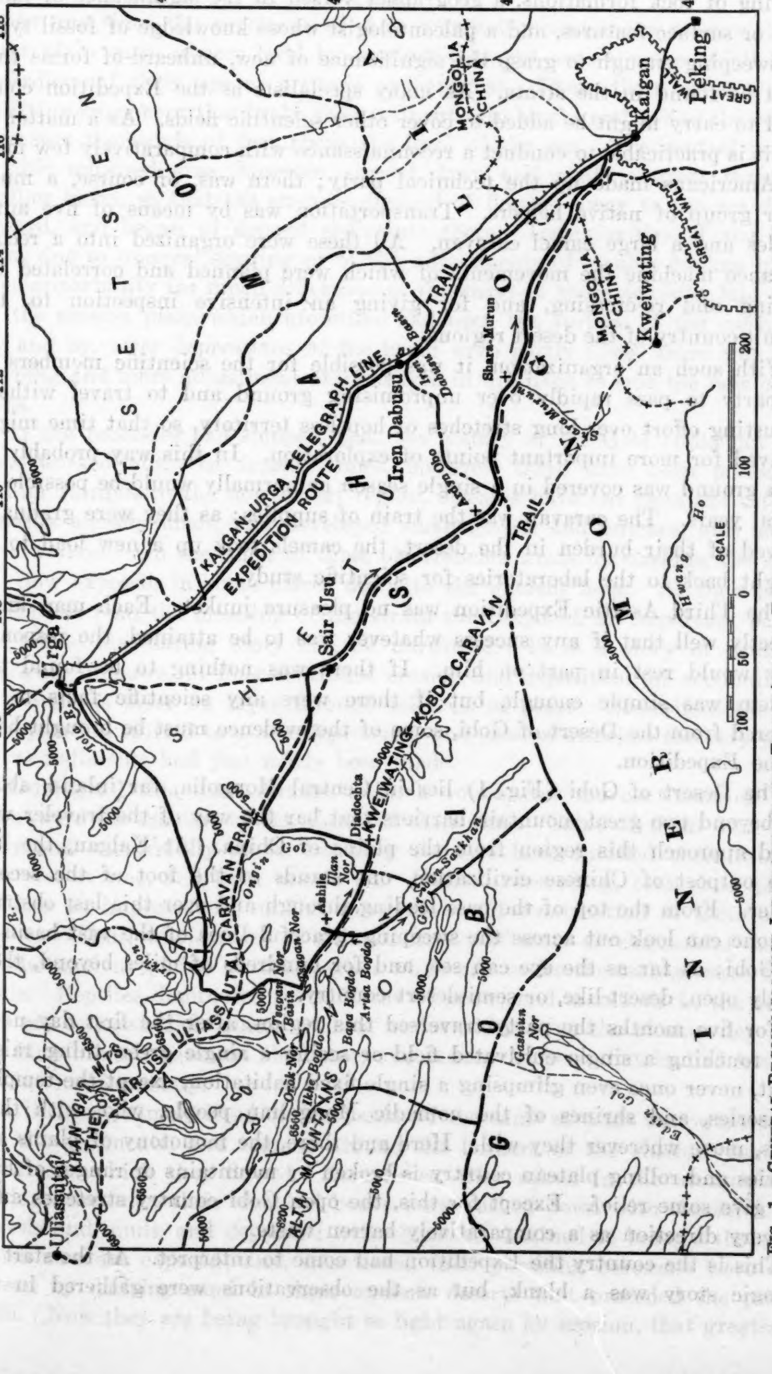


FIG. 1. GENERAL RECONNAISSANCE ITINERARY OF THE THIRD ASIATIC EXPEDITION, SEASON OF 1922, COVERING A TRAVERSE OF MORE THAN 3 000 MILES IN CENTRAL MONGOLIA.

If this was to be a comprehensive scientific reconnaissance, it was evident that the party should include a geologist experienced enough to interpret the meaning of rock formations, a geographer versed in the significance of landscape or surface features, and a paleontologist whose knowledge of fossil types was sweeping enough to grasp the significance of new, unheard-of forms that might be found in the strata. As many specialists as the Expedition could afford to carry might be added to cover other scientific fields. As a matter of fact, it is practicable to conduct a reconnaissance with comparatively few men. Six Americans made up the technical party; there was, of course, a much larger group of native helpers. Transportation was by means of five automobiles and a large camel caravan. All these were organized into a reconnaissance machine the movements of which were planned and correlated for crossing and re-crossing, and for giving an intensive inspection to, the difficult country of the desert regions.

With such an organization, it was possible for the scientific members of the party to pass rapidly over unpromising ground and to travel without exhausting effort over long stretches of hopeless territory, so that time might be saved for more important points of exploration. In this way probably as much ground was covered in a single season as normally would be possible in several years. The caravan was the train of supplies; as they were gradually relieved of their burden in the desert, the camels took up a new load to be brought back to the laboratories for scientific study.

The Third Asiatic Expedition was no pleasure junket. Each man knew perfectly well that if any success whatever was to be attained, the responsibility would rest in part on him. If there was nothing to be found the problem was simple enough, but if there were any scientific facts to be gathered from the Desert of Gobi, some of the evidence must be brought back by the Expedition.

The Desert of Gobi (Fig. 1) lies in Central Mongolia, far inland, above and beyond two great mountain barriers that bar the way of the traveler who would approach this region from the plains of China. At Kalgan, the last large outpost of Chinese civilization, one stands at the foot of the second barrier. From the top of the pass leading through and over this last obstruction one can look out across the sweeping, graceful lines of the vast basin of the Gobi; as far as the eye can see, and for hundreds of miles beyond, there is only open, desert-like, or semi-desert country.

For five months the party traversed this region, after the first day never once touching a single cultivated field or seeing a single green thing raised to eat, never once even glimpsing a single fixed habitation, except the temples, lamaseries, and shrines of the nomadic Mongolian people, who, with their flocks, move wherever they will. Here and there, the monotony of plains and prairies and rolling plateau country is broken by mountains or ranges of hills that give some relief. Except for this, the open Gobi country stretches away in every direction as a comparatively barren waste.

This is the country the Expedition had come to interpret. At the start its geologic story was a blank, but as the observations were gathered in the

course of the summer, they finally organized themselves into a connected story of the origin and history of the region.

At one time, the continent of Asia was not mountainous as now, but a great plain worn almost level by erosion, that cut across rocks of all kinds regardless of their age or attitude or hardness. Then, there came a great uplifting movement that broke the continent into blocks and raised the central segments thousands of feet above the level of the sea. The central portion of this upraised area, however, sagged down and made a great spreading shallow basin, so that the streams, instead of flowing away to the sea flowed inward, and began to deposit sediments there. Irregularities of the basin floor, due to unequal sagging or to the adjustments of smaller broken blocks, gave opportunity for much difference of thickness in these deposits, laid down on the erosion plain which aforetime had been the surface of the continent. By and by, after depressions of the basin were partly filled, there was more warping and more adjustment and changes in drainage to suit the new conditions.

If the processes had stopped then, the history of the Gobi could not have been worked out. In former times, however, there have been periods of greater rainfall in the now desert region, streams have cut channels into the deposits already made, and developed small gorges and gulches, and narrow river valleys. On the edges of these old trenches made by streams, the strata are now exposed, in some places down to the ancient floor. At such places, if one can read the meaning of the strata thus exposed, an important part of the whole fascinating story can be unraveled. It is a story of arid and changing climates, of long-continued erosion and deposition, of accumulation of sediments layer on layer through millions of years. Sometimes the area of deposition shifted to a new depression, and erosion attacked the very places where sediments had just newly been made.

During all this time, strange animals roved over the continent of Asia. They were the masters of the Earth of their time. Some of them were fearfully and wonderfully made, but they seem to have fulfilled their mission, whatever it was, and have passed on. Others, succeeding, took their places, perhaps an improvement on the appearance and construction of their predecessors. Thus, whole races of animals lived out their developmental destiny on the ancient plains of this region while the rocks themselves were being made. Reptiles dominated for millions of years, and mammals in the course of time usurped their places. Then, at last, almost at the closing day of it all, Man himself came on the scene, and it may be here that he had his first revelation of mastery.

All these animals lived on the banks of the same streams that were carrying the sediments, and roamed around the borders of the shallow ponds or lakes into which they flowed. Sometimes, they mired in the accumulating muds; sometimes, their bleached bones were washed down with the sands and gravels and muds, and deposited with them, all destined to become the strata which now are exposed; and, sometimes, their parching carcasses were overspread by drifting sand in these ancient deserts that preceded the present Gobi. Now they are being brought to light again by erosion, that greatest of

excavating processes, and left where it is perfectly easy to find them. At many different places the Expedition did find them—the dinosaur, the rhinoceros, the baluchitherium, and even flies, mosquitoes, and butterfly wings—a treasure house indeed. Tons of such material were brought back, and fields were located which will yield rich harvests for many succeeding years.

Some of these are forms the like of which have never been found before. These fill gaps in the tree of life. They are links long missing in the story of evolution with which all life of every age has been connected. Others are so evidently similar to forms previously found either in America or Europe that they seem to support the theory of frequent migrations, and thus substantiate the claim made long ago that there must have been land connections between them. Thus, one gets a glimpse of an inter-relation and interdependence almost worldwide in extent, and begins to appreciate that the discoveries in Central Asia form a vital part of the incomplete record of the story of life.

Each different life form has a surprisingly definite stratigraphic position in the geologic column. Those found in the older strata are more primitive and simple, as if in the relation of ancestors to those of later time. The strata in which they are buried, have been formed from one age to another, one on top of the other, throughout Upper Mesozoic and Tertiary time. By assembling the data of many different places, nearly all parts of this column are represented.

Thus, it becomes practicable to arrange the whole series and to sub-divide this complex assemblage of strata into definite formations and equally definite life zones. In this way, in a region where there was only "sand" and "rock" before, there has come to be an intelligible geological structure with a definite readable history, making one of the larger contributions to the understanding of the geological story of the earth.

Mile by mile of the traverse was charted to scale. Distances were recorded, and the aneroids were read for every important change of level, so that when all these readings were plotted, they yielded a profile of the country. Meanwhile, every outcrop of rock was scanned and if possible its significance determined with dip and strike, so that when these data were added to the profile, it grew into a continuous record of the structure of the country. The notebooks, of which Figs. 2 and 3 are samples, cover more than 3 000 miles of such geologic section; not a mile was slighted. Wherever possible, route maps were made mile by mile in the same systematic manner, and when the Expedition stopped for a longer time, special studies of selected areas were made in even greater detail. These have become the key studies of the Expedition and form the basis of final determinations in the structure and stratigraphy of the region, the skeleton or framework around which the more elaborate structure is built. Thus, one finds indications of the character of the country in those ancient times, with its climate and its life.

On the basis of all the evidence, there is practical certainty of what hitherto was surmised, that Northern Asia has been a continent capable of supporting land animals under favorable circumstances ever since Jurassic time. It has been a region of remarkable stability throughout the age of reptiles, and the subsequent ages of mammals and man. During all this time,

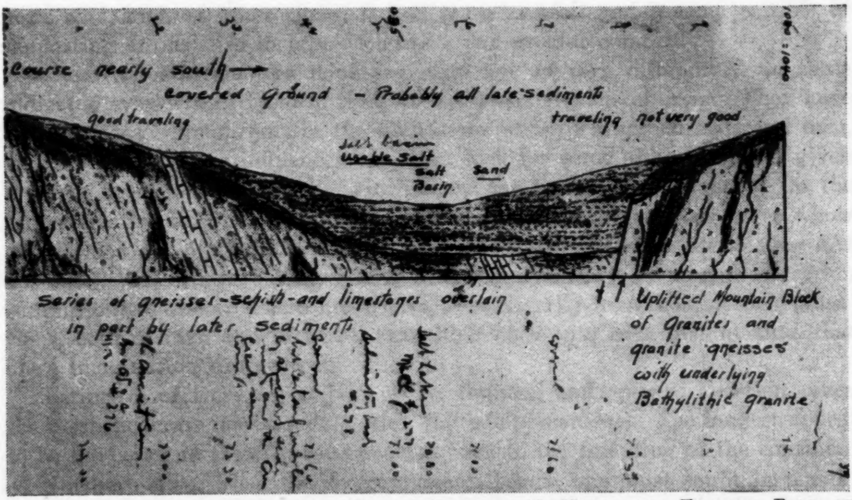


FIG. 2.—SPECIMEN OF GEOLOGICAL NOTES, COVERING 10 MILES AND FORMING PART OF CONTINUOUS CROSS-SECTION. DEPICTS SEDIMENT-FILLED BASIN, CONTAINING SMALL SALT LAKE. RELATION OF OVERLYING SEDIMENTS TO ANCIENT CRYSTALLINE ROCK FLOOR IS CHARACTERISTIC.

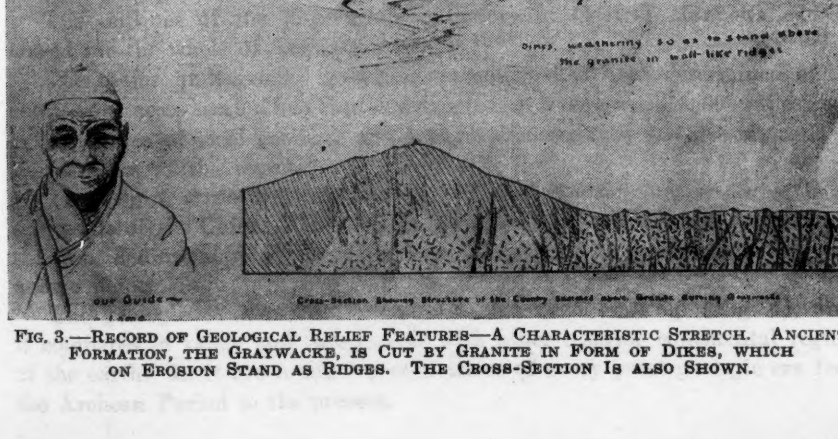


FIG. 3.—RECORD OF GEOLOGICAL RELIEF FEATURES—A CHARACTERISTIC STRETCH. ANCIENT FORMATION, THE GRAYWACKE, IS CUT BY GRANITE IN FORM OF DIKES, WHICH ON EROSION STAND AS RIDGES. THE CROSS-SECTION IS ALSO SHOWN.

open, semi-arid regions prevailed. Climatic changes came, not unlike those described by Huntington in his "Pulse of Asia". In certain epochs, it may have been colder and in others, a little warmer than now; sometimes, it was more arid, and, at other times, it had more rainfall and became capable of supporting animal life in larger numbers and greater comfort.

These climatic changes doubtless were not so very different from those indicated even within historic time in the desert regions of Asia. They have been immensely important in the migrations of both lower animals and men, and perhaps to those influences one must look for some of the impetus given to their development as well. How much the human race may owe to the prehistoric hardships of a changing environment that sometimes drove whole populations out of their Garden of Eden, perhaps will never be known. At any rate, it is certain that there have been repeated waves of human migration coming out of Asia. There must have been literally hundreds of such pulsations, both of man and of beast, stretching back into such distant time that one's imagination can scarcely follow.

Estimates of time are at best poorly founded and open to question; even the most moderate ones outstrip one's full comprehension. The longest appear to be the best and the span that reaches back to the founding of the continent of Northern Asia, when the desert climates began, one must count in tens of millions of years.

Whether or not it is worth while to determine any of these things is not for us to say. In making scientific discoveries of any kind, it is difficult to see their boundaries. The bearing of many observations the significance of which was not appreciated while in the field, is just coming to be understood—and it is possible that as these studies progress, the limits of their application will reach to still more distant fields and their value be more evident. No attempt has been made to discuss these questions here, but a summary of this work is included in the geologic chart, Fig. 4.

Perhaps it is sufficient to indicate the nature and scope rather than the meaning of some of the contributions of this Expedition. In a region the geology of which was almost wholly unknown, the following foundations of knowledge have been laid:

The outlines of the physiographic history of Central Mongolia can be traced for the whole of Tertiary time.

The major underground geological structure had been determined and a continuous cross-section has been constructed on a traverse of 3 000 miles; and 700 sq. miles of local geologic and topographic mapping have been made as type studies of the region.

The origin, structural relation, and ages of the different series of basin sediments of the Central Gobi region have been determined, and no less than thirteen definite formations have been distinguished where only one was known before.

The geologic column for Central Asia has been built up (Fig. 4), until it compares in range and definiteness with other interior continental regions of the earth. In it are found representatives of every great geologic era from the Archean Period to the present.

GENERALIZED GEOLOGIC COLUMN FOR CENTRAL MONGOLIA

Former Class'n		Classification by Geologists of Third Asiatic Expedition				
		Era	Series	Period	Formation	Chief Index Fossils
Not Subdivided	Not Subdivided	Cenozoic	Quaternary	Pleistocene	Olan and Diske (Andersson, 1923)	Elephant, Rhinoceros
			Upper Tertiary; Tsagan-nor Series	Pliocene	Hung Kureh	Deer, Mastodon, Hipparion, Struthiolithus (Giant Ostrich)
				Miocene	Loh	Trilophodon (A Primitive Mastodon)
Gobi Series (Obruchev) Considered all as Tertiary	Khan-khai Series (von Richthofen) Considered all as Tertiary		Lower Tertiary	Oligocene	Hsanda Gol Houldjin Ardyn obo	Baluchitherium, Rodents Baluchitherium, Entelodon Cadurcotherium (A Rhinocerid)
				Eocene	Shara Murun Irden Manha Gashato	Protitanotherium Desmatotherium (Primitive Tapir) Primitive Mammals
		Unconformity				
		Mesozoic	Late Mesozoic; Shamo Series	Upper Cretaceous	Iren Dabasu Dja-doch-ta	Trachydonts, Dinosaurs Protoceratops (Dinosaur) and Eggs
				Lower Cretaceous (Comanchean)	Ondai-sair Ashile	Protiguanodon, Sauropods, Fish, Fossil Mosquito, Etc., Psittacosaurus, Sauropods
		Great Unconformity All Rocks below this line are folded				
Not hitherto Subdivided			Early Mesozoic; Tsetsenwan Series	Jurassic	A Great Series of Conglomerates, Sandstones, and Shales, with Associated Lava Flows, Tuffs, and Ashes, carrying obscure Plant remains and locally, Coal, the whole about 20,000 Feet thick. Apparently corresponds to lower Jurrassic of Northern China.	
		Unconformity				
		Paleozoic	Late Paleozoic; Sair Usu Series	Permian Carboniferous	Limestones Shales Sandstones Slates Quartzites Conglomerates	A Series of Characteristic Invertebrate Fossils.
		Unconformity covering Early Paleozoic Time				
		Great Bathylithic Invasion				Mongolian Granite Bathylith
Not hitherto recognized in Mongolia		Proterozoic	Late	Nank'ou System A Name proposed first by von Richthofen and given definite Rank by Willis in China	The Sinian System of Grabau; The Khangai Series	The Tola River Graywackes and Slates without Fossils
			Early	Wu-T'ai System As used by Willis in China		Schists Phyllites Limestones Dolomites Quartzites Greenstones
		Archeozoic	Archean	The T'ai-Shan Complex As used by Willis in China		Crystalline Limestones Schists and Complex Injection Gneisses

FIG. 4.

Many geologic links have been filled in. The order of events with the processes and movements that have been prominent in the making of North Central Asia has been organized into the semblance of a connected geologic story. This is at least a beginning to a fuller understanding of the part played by this very ancient land, the influence of which appears to have been greater in former times than its present rather barren condition suggests.

Fossil-bearing sedimentary strata, of ages not previously known to exist in Central or Eastern Asia, have been found, thus adding important new fossil fields for vertebrate forms. These bring into view a whole continent where the development of life in ancient times parallels, supplements, and extends what is already known in other regions.

A large number of fossil forms new to science and many rare types have been recovered, so that these fields bid fair to surpass the most famous fossil fields of the world.

IMHOFF TANKS—REASONS FOR DIFFERENCES IN BEHAVIOR*

By Harrison P. Eddy,† M. Am. Soc. C. E.

Introduction

The object of the investigation reported in this paper was to ascertain, in so far as possible from available information, the reasons for the differences in operating results obtained from Imhoff tanks, in order that the requirements for satisfactory results might be better understood. The plants at Schenectady, N. Y., Plainfield, N. J., Fitchburg, Mass., and Rochester, N. Y., have been selected for comparison, for the following reasons: Two of the plants have functioned in a normal manner and two have developed serious difficulties; the operating data are unusually complete and reliable; and all the plants have had close, intelligent supervision.

Theory of Operation of Imhoff Tanks

There are two primary functions of Imhoff tanks: (1) the deposition, in the upper or sedimentation compartment, of suspended solids contained in the sewage; and (2) their conversion into inoffensive, relatively stable matter—a process commonly referred to as digestion—in the lower, or digestion compartment.

The degree of removal of suspended solids from sewage depends primarily on the length of the detention period, although tank efficiency may vary greatly, according to operating conditions.

It is possible that the more prolonged detention period provided in some sedimentation compartments, resulting in greater removal of solids, has had an indirect influence on the process of converting the solids into stable matter, because of the removal from the sewage of a larger proportion of the finer particles, which are more susceptible to digestion.

The principal difficulties experienced in the operation of Imhoff tanks have been connected directly with the action going on in the digestion compartments. This part of the tank may be subdivided into sludge and scum compartments, separated only by an imaginary horizontal plane passing through the slots. It should be recognized that sludge must not be allowed to rise to within less than 18 in. of the slots, and that scum should not extend downward to within that distance above the slots. For convenience, these parts of the sludge and scum compartments may be called the "neutral zones".

Solids that pass through the slots may be divided into two classes, namely, those of greater specific gravity than the sewage, which settle to the bottom,

* Presented at the meeting of the Sanitary Engineering Division of the Society, January 17, 1924. Written discussion on this paper will be closed with the September, 1924, *Proceedings*. When finally closed the paper, with discussion, will be published in *Transactions*.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

and those of less specific gravity, which float in the scum compartment. The proportion of solids of the latter class is small. Such solids comprise substances which have been entrained in the mass of heavier solids and thus caused to settle.

The solids which are removed from the sewage by sedimentation are, with the exception of sand and mineral detritus from storm water, and certain industrial wastes, essentially organic, although such organic matter contains mineral ingredients. These organic substances are, or soon become, decomposed owing to the action of bacteria and other living organisms, or their enzymes, present in the sewage and propagated in the digestion chamber.

In time, these organisms decompose the organic matter into gases, soluble substances, and insoluble material. It is true that some kinds of organic matter are more easily and more quickly decomposed than others. The finely divided solids, which are more largely removed in tanks providing long detention periods, probably belong in the former class.

Of the products of decomposition, the gases escape by solution in the liquid and by passage through it into the atmosphere; the soluble substances are dissolved in the liquid in the digestion compartment, from which they escape either by diffusion through the slots or by withdrawal of the sludge (in either case, the liquid is replaced by sewage from the sedimentation compartment); the stable solids remain as sludge and scum in the digestion compartment. It appears, in general, that from 30 to 50% of the weight of solids removed from the sewage escape in solution or as gas, leaving 70 to 50% as solids in the sludge and scum.

The digestion of the organic matter results in the production of a practically inoffensive residue, or sludge, which is generally black, somewhat flocculent, well filled with gas as drawn from the tank under ordinary conditions, and drains and dries readily when spread on porous beds of sand or similar material. Digestion sufficient to produce these characteristics may be considered complete for practical purposes, although this process may be carried further.

Time is required to complete the conversion of the fresh sewage solids into well digested sludge, after the ripening process has once become established, the length of time being greatly influenced by temperature. If the sludge is kept at a temperature just above freezing, scarcely any digestion will take place; if heated to 70° Fahr., digestion will proceed rapidly.

Theoretically the well digested sludge should lie at the bottom of the accumulation and the fresh sludge at the top, there being a gradation in quality between these two extremes. Therefore, when sludge is withdrawn, it should be taken from the bottom.

In northern climates such as those in the four cities under consideration, there is a great variation from season to season in the temperature of air, sewage and, most important of all, solids and liquids in the digestion compartment. In summer, when the temperature in the sludge compartment may be from 65° to 70° Fahr., digestion is rapid, as indicated by vigorous evolution of gas, but, in winter, it is greatly retarded. The digestion process follows a seasonal cycle. In summer, the sludge may be in excellent condition, deteri-

orating during the fall, becoming quite offensive in winter, and gradually improving in the spring. In some cases, sludge withdrawn in winter is of poor quality and quite offensive. Obviously, therefore, the digestion compartment must be of sufficient size to accommodate the solids deposited in the late fall, winter, and early spring, so that it will not be necessary to withdraw partly digested and offensive sludge.

The solids as first deposited form a thin, voluminous sludge. As decomposition progresses, they are disintegrated and, being finer, tend to become compact. This tendency is aided by the weight of solids subsequently deposited. As a result of these conditions, the mass varies in consistency from a thin, watery material at the top to a comparatively thick sludge at the bottom.

Gases generated during the decomposition of the solids gradually increase in volume until they permeate and buoy up masses of sludge which pass into the overlying liquid. If there is opportunity for the gases to escape from the solids in which they are entrained, the solids will return to the sludge mass, because of their relatively high specific gravity. In practice, however, the gases escape with some difficulty, and masses of sludge are carried into and remain in the scum compartments, thus forming a heavy scum of very much greater volume than those floating solids which are actually of lighter specific gravity than the sewage. If the sludge or the scum becomes extremely compact, there is not opportunity for the proper functioning of the organisms, and the digestion process is greatly retarded and may be quite incomplete.

Difficulties in Operation

At some plants much of the gas rising in the vents has produced a light, voluminous foam, due to tenacious films enclosing the gas bubbles. At times, this foam has accumulated in considerable masses, filling the gas vents, overflowing their walls, and spreading out and covering walks and the sewage in the sedimentation compartment.

Excessive accumulation of gas-lifted solids—scum—in the gas vents, has been another serious obstacle to successful operation. In some plants, the scum has risen above the tops of the gas vents and, in others, it has extended downward practically to the elevation of the slots. Under the latter condition, solids have been carried upward through the slots into the sedimentation compartment, from which they escaped with the settled sewage, greatly decreasing the efficiency of sedimentation.

Under some conditions the proportion of solids remaining in the bottom of the tank has been so small that it has been practically impossible to remove any solids in the form of sludge without first driving down the scum by breaking it with paddles or streams of water to liberate the gas. If scum is thus broken and the solids are allowed to settle, they rise again in a comparatively short time.

The reduction of the scum by hand methods or by hose streams, involves a comparatively heavy operating expense. Mechanical agitators are said to have been successfully used for breaking up scum at Newton, Kans., and at Enid, Okla.*

* *Engineering News-Record,* August 23, 1923, p. 301.

In some plants difficulty has been experienced in obtaining an inoffensive sludge that will dry rapidly on sludge-drying beds. Offensive odors have escaped from the Imhoff tanks at some plants, although generally speaking these tanks are practically free from odors.

Comparative History of Plants

At Schenectady, the Imhoff tanks first operated in January, 1915, have never given entirely satisfactory results and have proved to be very expensive to operate. At times, they have produced great quantities of foam which overflowed the gas-vent walls, the free-board being originally slightly less than 12 in. There has never been much foaming during cold weather. The introduction of lime into the sludge compartment seemed, sometimes, to reduce, and at other times, to increase, the tendency to foam. In all instances, it appeared to modify biological action in the sludge in such a way as to make it offensive.

During the earlier years, the foam was prevented from overflowing the gas-vent walls by playing a hose stream on the scum in the vents. This operation required much labor and at times seemed to increase the foaming. During 1916, the walls of the gas vents were increased about 14 in. in height by means of wooden boxes, which provided a free-board of approximately 26 in. The sludge riser pipes terminate in an open bell about 18 in. below the top of the boxes, and in the center of the gas vents. Since the gas vents were raised, foam has been drawn into the sludge risers, from which it flowed to the sludge drains, when necessary to prevent the foam from overflowing the gas vents. This has resulted in a great reduction in the labor required to prevent foaming. Frequent removal of sludge from the digestion chambers has also been of assistance in preventing foaming.

There has always been a tendency for large quantities of scum to form in the gas vents. Some of this scum has been removed through the sludge riser pipes and, at times, has been comparatively free from odor; at other times, it has been offensive. During 1922, the scum was allowed to accumulate throughout most of the year, in order to determine if such accumulation would prevent foaming, and if digestion or other actions would finally cause the solids to settle into the sludge compartment. Foaming was practically eliminated by this treatment, but the solids continued to form scum until it extended down practically to the elevation of the slots, after which solids escaped through the slots into the sedimentation compartments in such quantity as greatly to reduce the sedimentation efficiency, to cause unsightly scum on the surface of the sewage in these compartments, and to increase the clogging of the trickling-filter nozzles due to the solids carried out of the tanks. The experiment was finally abandoned, some scum being shoveled out of the gas vents, and some being broken by a self-propelling nozzle attached directly to fire hose. The sludge in the tanks finally was pumped out with the exception of a few hundred gallons left in each hopper for the purpose of seeding the incoming material when the tanks were replaced in service.

After the tanks had been again put into operation, an attempt was made to prevent the solids from remaining indefinitely as an accumulation of scum.

As soon as scum began to form in considerable quantity, the gas vents of the tanks, with one exception, were attacked by hose streams to break up the scum and to liberate the gas, after which the solids promptly settled into the sludge compartments. Although further gas generation gradually caused the formation of scum, it was possible to draw sludge from the digestion compartment for one to two days after the hose treatment. At the end of three days, practically all the solids again rose into the scum compartments.

The labor required for liberating the gas from the scum, whether by use of paddles or hose streams, and, in fact, any work required in the gas vents, is exceedingly laborious because of the large number of such vents, 16 per tank, or 144 for the whole plant, which treats about 6 000 000 gal. of sewage daily.

The scum became very dense and compact if allowed to accumulate for a long period, a condition unfavorable for digestion. During 1923, when the scum was broken up frequently throughout the warmer season, digestion appeared to proceed moderately, but not vigorously. The sludge removed was not extremely offensive, although it was not free from objectionable odor. It contained a comparatively small proportion of solids and showed that digestion had not progressed to the necessary degree of completion.

The sludge at Schenectady has always been comparatively thin and has not dried as well as at some other plants. During the summer and early fall, it has usually not been offensive; during the winter and early spring, however, digestion appears to have been reduced to such an extent that the sludge is exceedingly offensive. Although there has been comparatively little odor about the tanks, the slight discoloration of paint on the wooden boxes is evidence that some hydrogen sulphide has been liberated.

At Plainfield the Imhoff tanks, first operated in November, 1916, have been subject to excessive foaming, and a great deal of scum has been removed from the gas vents in order to maintain the plant in efficient and presentable condition.

During the early years of operation, foam overflowed the gas-vent walls and at times covered the sedimentation compartments to a depth of 1 ft., or more. Continuing during at least one winter, it rose in the gas vents, and froze in conical mounds in the form of stalagmites that extended 4 ft. above the gas vents.

Numerous methods to prevent foaming were tried, but none was entirely successful. After about two and one-half years of operation, a Riensch-Wurl screen with $\frac{1}{16}$-in. slots was provided for screening the incoming sewage. At the same time, the method of operation was changed from that of using all the tanks at once to only three at a time, in order to increase the velocity of flow through those used, and thus obtain a more uniform distribution of sludge in the several hoppers.

The remaining three tanks were kept detached, in order to allow digestion to proceed undisturbed. The tanks were operated in rotation, the schedule being such that one was removed from service and one placed in action each day, thus allowing each tank to receive sewage for three days, and then to be quiescent for three days. Although foaming has been reduced since these changes were made, it is still much more prevalent than at some plants, and

as a preventive measure, it is necessary to remove scum from the gas vents by manual labor. It is said that objectionable odors have been traced to the Imhoff tanks. The sludge has always contained a comparatively small proportion of solids and has not dried as rapidly as would be expected with a normal sludge.

At Fitchburg, the Imhoff tanks, which were first operated in October, 1914, have been free from persistent foaming and excessive scum formation.

During August, 1915, Tank No. 1, which had been in continuous use since the previous October and had had some sludge withdrawn, became very active. Foam rose to a maximum height of 4½ ft. above the sewage level in the central gas vents which are about 5½ ft. high but did not appear in the side vents. Shortly before this, humus tank sludge had been introduced into this unit. The sewage was shut off from this tank, in order to make repairs, for about two months. After operation was resumed, there was no further trace of foaming throughout the year. Another tank which had been operated in parallel with Tank No. 1, showed no signs of unusual activity.

During June, 1916, foam rose 18 in. in the east row of side vents of Tank No. 1, and overflowed into the sedimentation chamber. Disintegrating the scum with a water-jet had no permanent effect, but the drawing of sludge caused the foaming to cease.

During August, 1917, after a week of very hot weather, there was some foaming in the central chimneys of Tank No. 5. The sludge compartment was found to be filled to the slots. After sludge was withdrawn, the foam subsided and action became normal. The dates on which foaming has occurred during the last five years are given in Table 1.

TABLE 1.—Data Relating to Foaming at Fitchburg, Mass.

Year.	Tank.				
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
1919	July 13 July 14	August 6 August 8	August 7 August 8	None	None
1920	None	June 14 July 2	June 14 July 15	None	None
1921	None	None	June 4	None	None
1922	None	None	None	None	None
1923	None	July 6	July 6 July 7	July 6	None

Generally, no effort was made to break up accumulations in the gas vents; on a few occasions, hose treatment was used.

Scum has not accumulated sufficiently to cause much trouble. A small quantity has been removed from a few of the gas vents, in order to reduce the pressure on the walls. The record of removal of scum for the last three years is as follows:

1921	48 cu. ft. from Tank No. 5.
1922	None
1923	36 cu. ft. from Tank No. 2 10 cu. ft. from Tank No. 3 40 cu. ft. from Tank No. 5

It is fair to state that difficulties due to foaming and to scum formation have been negligible throughout the nine years of operation. It appears to be possible to prevent excessive foaming by restricting the accumulation of sludge and to stop it by drawing sludge.

There has not been any odor from the tanks except that due to the exposure of the fresh sewage to the atmosphere. The sludge at Fitchburg when drawn has been practically free from objectionable odor and has contained a comparatively large proportion of solids. When applied to the sand beds, it has dried readily.

At Rochester, the Imhoff tanks of the Irondequoit Plant, first operated during March, 1917, have only once given evidence of excessive foaming. This occurred in a single tank that had been allowed to become filled to the slots with sludge. The scum in the gas vents has been removed from time to time, so that it has never risen much above the level of the sewage in the sedimentation compartments.

There has been no odor about the tanks other than that to be expected from exposure to the atmosphere of the sewage in the sedimentation compartments. The sludge has contained a large proportion of solids, has been free from objectionable odor, and has dried rapidly.

General Comparison

Although equally complete data are not available for all the plants, some information regarding the density of the sludge produced may be obtained from Table 2.

TABLE 2.—Solids in Sludge as Drawn

Plant.	Period.	Percentage of solids.
Schenectady, N. Y.	1918	3.3–7.3*
Plainfield, N. J.	November 21, 1922 April 30, 1923	7.1 (average)
Fitchburg, Mass.	1922	12.85 (average)
Rochester, N. Y.	....	14-21

* Exclusive of one abnormally low determination.

In comparing the figures in Table 2, it should be noted that the Plainfield sludge was drawn during the winter and may not have been in as good condition as during the summer season. The sludge drawn at Schenectady and Plainfield appears to contain a much smaller proportion of solids than that from the other two plants. If it is fair to assume 7.5% solids for the former and 15% for the latter, the volume of sludge drawn at Schenectady and Plainfield would be twice as great as that at Fitchburg and Rochester for the same weight of solids.

Data are not available for an accurate comparison of the time required for the several sludges to dry to a definite extent; there have been instances at each plant of drying in as short a period as 10 days for a depth of application of about 10 in. In general, sludge beds are filled from six to ten times per year.

Consideration of all available information leads to the conclusion that the Schenectady and Plainfield plants have never functioned in a fully normal

manner, as those at Fitchburg and Rochester. It should be remembered, however, that some foaming has occurred on occasions and that scum has formed to some extent at the Fitchburg and Rochester plants. These facts indicate that similar underlying conditions exist at these plants and also emphasize the possibility that there are certain differences, perhaps only in degree, which, if discovered, will explain the divergence in results obtained.

The possible causes of differences in performance of tanks at the several cities will be considered under five headings, as follows:

Character and Composition of Sewage;
Temperature of Digestion;
Preparatory Treatment of Sewage;
Kind of Biological Action;
Tank Design; and
Method of Operation.

Character and Composition of Sewage

The sewages at Schenectady and Plainfield are from separate systems, whereas those from the other cities are from combined systems.

Sewage from combined systems contains considerable quantities of solids washed by rains from the streets into the sewers. Grit-chambers have generally been provided with combined systems in order to prevent the heavy mineral detritus from entering sedimentation tanks. Such chambers vary greatly in efficiency according to their design and operation. Grit-chambers usually remove much organic and mineral detritus, but probably all allow considerable material washed from the streets to pass into the tanks. At Fitchburg, this condition was so prevalent that it has been necessary to remove accumulations from the tanks.

The fine mineral detritus and the organic matter from the streets, when mingled with the sewage solids in the sludge compartment, may tend to weight them down and thus prevent scum formation to the same extent as in tanks receiving separate sewage. It is probable also that the volume of gas produced per pound of solids deposited is less for combined sewage. Other conditions being the same, both these facts would tend to explain the more favorable behavior of the tanks at Fitchburg and Rochester.

There does not appear to have been much difference in the comminution of suspended solids at Schenectady, Fitchburg, and Plainfield. Since the installation of fine screens at Plainfield such matter has largely been prevented from entering the tanks. The difficulties due to formation of scum which appears to be favored by a large proportion of uncomminuted solids have been troublesome at Schenectady and Plainfield, and practically negligible at Fitchburg.

The sewage reaching the plants at Schenectady and Fitchburg is comparatively fresh, possibly somewhat less so at Fitchburg, where the sewage passes through a 30-in. inverted siphon, 5 300 ft. long, in which some solids are deposited. Decomposition of organic matter takes place to some extent in the siphon. At Plainfield and Rochester, the sewage is stale. The Schenec-

tady and Plainfield plants have proved troublesome although they treat fresh and stale sewage, respectively. The Fitchburg and Rochester plants receive fresh and stale sewage, respectively, and yet practically no difficulty has been experienced at either.

Industrial Wastes.—Although at times the Fitchburg sewage has contained small quantities of paper-mill, wool scouring, and metal trades wastes, industrial wastes do not appear to have been present in sufficient quantity in this or any of the other sewages to affect greatly their composition.

Effect of Quality of Water Supply.—At Schenectady and Plainfield, the water supplies are comparatively hard. At Rochester, the water is nearly as hard as at Plainfield, and, at Fitchburg, it is distinctly soft.

The hardness of a water supply results in a large consumption of soap and in the formation of a correspondingly large proportion of insoluble soap or soap precipitate, as indicated by the results of a theoretical computation given in Table 3. These figures are based on the assumption that the same per capita daily volume of water is softened in all cases. The estimated weight of insoluble soap produced varies from 5 to 25 parts per million and is equivalent to 2.7 to 15.8% of the suspended matter in the sewage.

TABLE 3.—PROPORTION OF INSOLUBLE SOAP IN SEWAGE.

	Schenectady.	Plainfield.	Fitchburg.	Rochester.
Sewage flow, in gallons per capita per day	92	85	89	123
Hardness of water supply, in parts per million	130	88	10	65
Water softened per pound of soap, in gallons*	36	51	210	67
Assumed volume of water softened, in gallons per capita per day	1.0	1.0	1.0	1.0
Soap consumed, in pounds per capita per day	0.0278	0.0196	0.0048	0.0149
Insoluble soap produced†, in pounds per capita per day	0.0195	0.0137	0.0034	0.0104
" " " " pounds per million gallons	211	161	38	85
" " " " parts per million	25	19	5	10
" " " " grammes, per capita per day	9	6	2	5
Total suspended solids, in sewage, in parts per million	163	166	219	163
" " " " " " grammes per capita per day	57	53	74	76
Proportion of insoluble soap in suspended solids based on parts per million	15.3%	11.4%	2.3%	6.1%
Proportion of insoluble soap in suspended solids based on grammes per capita per day	15.8%	11.3%	2.7%	6.6%

* Based on Table 5, "Value of Pure Water", G. C. Whipple, p. 26.

† 0.7 lb. per lb. of soap.

The soap precipitate in sewage is finely divided and of light specific gravity. It may be a factor in the production of foam and scum. In cities supplied with hard water, it obviously adds greatly to the normal quantity of suspended solids to be removed from the sewage and digested.

Strength of Sewage.—The strengths of the sewages, measured in parts per million of suspended solids, are remarkably similar at Schenectady, Plainfield, and Rochester; at Fitchburg, the sewage is considerably stronger than that at the other plants. (Table 4.)

The suspended solids average 55 grammes per capita in the separate sewages and 75 grammes per capita in the combined sewages. The combined

sewages contain 20 grammes per capita, or 36.4% more suspended solids than the separate sewages, which may be accounted for by detritus and other matter washed from the streets during storms. This proportion is in close accord with a number of estimates made in the past.

The quantity of suspended solids in the sewage, however, is not the only consideration, the proportion of these solids removed by sedimentation being equally important. The proportion removed varied from 44% at Rochester to 76%* at Fitchburg. Although the Fitchburg sewage contained 32% more suspended solids, in parts per million, than the Plainfield sewage, the parts per million of solids deposited in the tanks were 88% greater. If the volumes of sewage and the tributary populations are taken into account, it will be seen that the Fitchburg contribution is 56 grammes per capita per day, or 93% greater than that at Plainfield.

TABLE 4.—Suspended Solids in Sewages and Deposited in Tanks.

Plant.	Year.	Population.	Flow, in million gallons per day.	Flow, in gallons per capita per day.	Suspended Solids in Sewage.		Percentage of suspended solids removed by sedimentation.	Suspended Solids Deposited in Digestion Chamber.	
					In parts per million.	In gallons per capita per day.		In parts per million.	In grammes per capita per day.
Schenectady	1922	65 000	6.0	92	163	57	71	115	40
Plainfield....	1922	40 000	3.4	85	166	53	54	89	29
Fitchburg..	1922	38 000	3.4	89	219*	74	76†	167	56
Rochester...	1922	260 000	32.0	123	163	76	44	73	34

* Due to some question as to accuracy of the figure of 266 parts per million obtained for 1922, 219 parts per million the average for the 6-year period (1915–1920, inclusive), has been substituted.

† This percentage is derived from the records for 1915 to 1920, inclusive, being substituted for the 1922 results which are open to some doubt, and includes the sludge from the humus tanks, which is pumped into the Imhoff tanks.

The wide difference between the number of grammes per capita per day of solids deposited in the tanks at Schenectady and Plainfield and at Fitchburg and Rochester forcibly illustrates the danger of basing the design of sludge compartments as well as other features of Imhoff tanks on a general allowance of a uniform number of cubic feet per capita. Such differences might be greatly increased by the presence of a substantial volume of certain kinds of industrial wastes in the sewage.

Temperature of Digestion

Temperature has an important influence on the rate of sludge digestion. Scarcely any determinations of temperature in digestion chambers have been made, the few available being given in Table 5.

In the absence of information on the rate of digestion in the sludge compartments of Imhoff tanks, data furnished by an experiment with a closed septic tank (Table 6) may be utilized to illustrate the effect of temperature on rate of digestion. It is recognized that this experiment was performed

* Including secondary tank sludge pumped back.

for another purpose and that conditions were different from those in the sludge compartment of an Imhoff tank; however, the general relation disclosed between temperature and biological action may properly serve to illustrate the principles involved in the Imhoff tank.

TABLE 5.—TEMPERATURE OF SLUDGE IN DIGESTION COMPARTMENTS OF IMHOFF TANKS

Plant.	Date.	Temperature, in degrees Fahrenheit.
Schenectady	Sept. 6, 1923	65 to 66
	Oct. 3, "	60
	Oct. 10, "	62 to 63
	Nov. 7, "	56
	Nov. 9, "	55
Plainfield	Sept., 1923	60 to 61
Fitchburg	Nov. 8, 1923	57

If the volume of gas is taken as the measure of digestion, such action was more than three times as rapid between 65° and 70° Fahr. as it was below 55° Fahr.

The relation of temperature to gas production at Worcester, Mass., is shown in Fig. 1. If this line is accepted, for the purpose of this discussion, as representing this relation, it appears that gas evolution practically ceases at 40° Fahr., that the mean annual rate of gas production is equal to that at 55° Fahr., and that at 70° Fahr. the rate of production is double the annual mean. It has been assumed that digestion proceeds at the same proportionate rate as gas production.

TABLE 6.—GAS EVOLVED FROM SEPTIC TANK*

Month.	Temperature of sewage, in degrees Fahrenheit.	GAS EVOLVED. Gallons per 100 gal. of sewage.	GAS EVOLVED. Percentage of annual mean.
January	48.2	1.17	30
February	47.0	2.45	62
March	46.7	1.87	48
April	49.2	2.00	51
May	55.4	3.98	100
June	59.5	5.84	148
July	64.2	5.52	140
August	66.4	6.62	168
September	66.6	6.69	170
October	61.2	4.55	116
November	57.0	4.53	115
December	49.3	2.57	65
Mean	55.9	3.94	......

* "The Action of the Septic Tank on Acid Iron Sewage", by Leonard P. Kinnicutt and Harrison P. Eddy, Fourth Report, Sewerage Commission of the State of Connecticut, 1902.

These data have been applied to the digestion of sludge in a hypothetical Imhoff tank assumed to receive from combined sewage 100 lb. of deposited solids per month, of which 60 lb. are assumed to be organic, and 40 lb.

mineral matter. It has been assumed also that 30 lb., or 50%, of the organic matter must be digested, in order to produce an inoffensive sludge. This is equivalent to 30% of the total solids. For convenience, this portion of the organic matter is designated "digestible solids" although digestion is not confined to it. With separate sewage, the proportion of digestible solids would approximate 40% of the total solids.

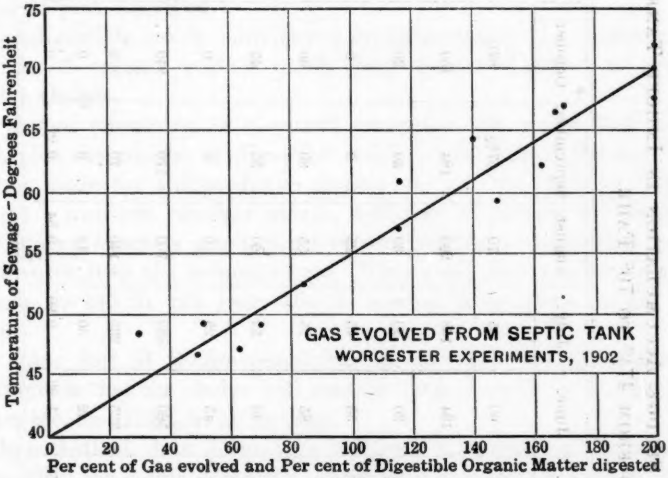


FIG. 1.

On these assumptions, the data in Table 7 show the progress of sludge digestion and accumulation at temperatures ranging from 47½° Fahr. during February and March to 70° Fahr. during August. These temperatures are probably as favorable as are likely to obtain in the northern part of the United States.

The digestion of digestible solids for the appropriate temperature is taken directly from Fig. 1, and expressed in percentage of the mean annual rate of digestion. The digestible solids deposited, are taken at a uniform quantity of 30 lb. per month. The residue of digestible solids from the previous months represents the quantity remaining undigested.

December is the first month, after the sludge has been completely drawn from the tank, in which there is a deficiency in digestion, such deficiency being 16% of the digestible solids deposited during that month, leaving approximately 5 lb. of residue of digestible solids on January 1. The total digestible solids subject to digestion during any month consist of the residue from the previous month plus the increment during the month.

The solids digested during the month are computed by applying the percentage digestion for the appropriate month to the digestible solids deposited in the tank each month. During months in which the digestion is 100%, or less, there is no decrease in the accumulation of digestible solids present in the tank on the first of the month. During months in which the digestion is in excess of 100%, this excess applied to the digestible solids deposited

TABLE 7.—THEORETICAL COMPUTATION OF SLUDGE ACCUMULATION IN IMHOFF TANKS AT TEMPERATURES RANGING FROM $47\frac{1}{2}°$ TO 70° FAHR.

Month.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
Temperature, in degrees Fahrenheit	50	47½	47½	50	55	60	65	70	67½	62½	57½	52½
Digestion of "digestible solids", percentage	68	50	50	68	100	134	168	200	184	150	118	84
Digestible solids deposited, in pounds	30	30	30	30	30	30	30	30	30	30	30	30
Residue digestible solids from previous month, in pounds	5	15	30	45	55	55	45	25	0	0	0	0
Total digestible solids in tank, in pounds	35	45	60	75	85	85	75	55	30	30	30	30
Solids digested in month, in pounds	20	15	15	20	30	40	50	60	55	45	35	25
Residue of digestible solids at end of month, in pounds	15	30	45	55	55	45	25	0	0	0	0	5
Undigestible solids contributed by sewage, in pounds	210	280	350	420	490	560	630	700	770	840	70	140
Sludge remaining in tank: Solids, in pounds	225	310	395	475	405	325	235	140	70	0	70	145
Cubic feet, average solids, 12½%	29	40	51	61	52	42	30	18	9	0	9	19
Cubic feet per capita	0.87	1.20	1.53	1.83	1.56	1.26	0.90	0.54	0.27	0	0.27	0.57

during the month gives the amount of solids digested from the accumulation of such solids in the tank at the beginning of the month. This method is adopted because the volume of gas produced in the Worcester tank is reported in terms of gallons per 100 gal. of sewage introduced into the tank during the month.

The residue of digestible solids at the end of the month is equal to the total digestible solids in the tank on the first of the month plus such solids deposited during the month minus the solids digested during the month computed as just described.

The undigestible solids contributed comprise the 40 lb. of mineral matter and 30 lb. of organic matter which need not be digested to produce an inoffensive sludge.

The sludge remaining in the tank comprises the undigested solids contributed, plus the residue of digestible solids at the end of the month, deduction being made for sludge drawn during six months, May to October, inclusive, at a uniform rate per month, sufficient to remove all sludge at the end of October, except a small quantity constituting a circulating load which does not enter into the computations. The solids drawn amount to 140 lb. per month, or 840 lb. per year, 360 lb. having disappeared as the result of digestion.

The cubic feet of sludge remaining in the tank has been computed on the assumption that the sludge will contain 15% of solids at the bottom, 10% at the surface, and 12½% as an average.

The hypothetical data have been reduced to terms in common use by assuming that the solids deposited in the tank correspond to 3 lb. per capita per month. For domestic sewage, this would approximate 2.2 lb. per capita, per month. This computation indicates a required capacity for sludge storage of 1.83 cu. ft. per capita, the maximum quantity being stored at the end of April. To this must be added an allowance for the "neutral zone" between the surface of the sludge and the horizontal plane through the slots, say, 20%, making a total required capacity of 2.20 cu. ft. per capita.

The total sludge drawn per year, assuming 15% solids, is equivalent to 2.715 cu. ft. per capita.

The rate of digestion and the per capita sludge accumulation, based on temperatures 5° lower than those used in computing Table 7, are given in Table 8.

This computation indicates that a decrease of 5° in temperature results in an increase of maximum sludge accumulation from 1.83 to 2.04 cu. ft. per capita, which, with a 20% allowance for the neutral zone, requires an increase in sludge storage capacity from 2.20 to 2.45 cu. ft. per capita, not allowing for storage of any solids in the scum compartment. This is an increase of slightly more than 10%, which would be greater if the difference between the assumed schedules of temperature were greater. It may be that in practice the differences are considerably greater than those assumed.

Curves *A* and *B*, Fig. 2, illustrate the variation in rate of digestion and evolution of gas, with temperature. The full line, Curve *B*, shows the rate of digestion, based on the assumption that sludge is drawn, thus reducing

the quantity of digestible matter in the tank and, therefore, also the rate of digestion; the dotted line shows the rate of digestion on the assumption that sludge is not drawn, therefore, the rate of digestion is greater than that shown by Curve *B*.

TABLE 8.—THEORETICAL COMPUTATION OF SLUDGE ACCUMULATION IN IMHOFF TANKS AT TEMPERATURES RANGING FROM 42½° TO 65° FAHR.

Month.	Temperature, in degrees Fahrenheit.	Sludge remaining in tank, in cubic feet per capita.
January	45	0.96
February	42½	1.32
March	42½	1.71
April	45	2.04
May	50	1.77
June	55	1.44
July	60	1.11
August	65	0.72
September	62½	0.33
October	57½	0.00
November	52½	0.30
December	47½	0.63

At the lower temperature (Curve *A*), there is always an excess of digestible matter. Therefore, the drawing of sludge does not affect the rate of digestion. Curves *C* and *D* show the rate of accumulation of sludge by volume at the different temperature schedules assumed, and the effect of regular uniform removal of sludge from May to October. The importance of beginning to draw sludge as early as inoffensive material can be obtained, and at a proper rate, is readily seen. The dotted lines, Curves *C* and *D*, show the rate of accumulation if sludge is not drawn.

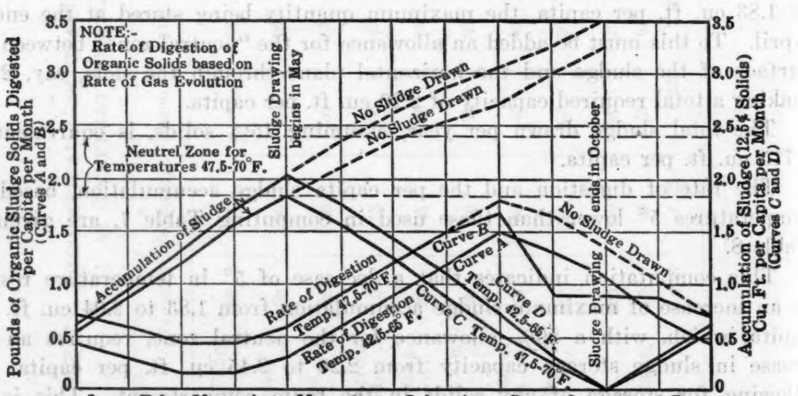


FIG. 2.—ACCUMULATION OF SLUDGE AND RATE OF DIGESTION OF ORGANIC SOLIDS IN IMHOFF TANKS.

It should be noted that, on the basis of the assumed schedule of operation, the maximum accumulation of sludge occurs at the end of April, and that the maximum volume of gas is generated at the end of August.

For the temperature schedule, 47.5° to 70° Fahr., the sludge chamber should be proportioned to accommodate 1.83 cu. ft. per capita of sludge below the neutral zone. Curve *D* shows that the sludge accumulation at 42.5° to 65° Fahr., will occupy nearly half of such a neutral zone.

It will be noted that, if the sludge is not drawn according to the assumed schedule, the neutral zone will soon become entirely filled and, also, that the excessive sludge accumulation will occur when gas is being produced at a high rate. This combination accounts for much of the foaming that has occurred at many plants. It is easy to understand also why the drawing of sludge frequently causes foaming to cease.

The temperature during the summer months is of more importance than the yearly average temperature, because during these months the largest quantity of solids must be digested. It is probable that an unusually low temperature in the cool part of the year, together with an exceptionally high temperature in a short warm season, may result in an abnormally large accumulation of digestible solids during the cool period, followed by very violent digestion during the warm period. This combination of conditions might bring about the excessive foaming that has occurred at various plants during the warm months. Winter foaming, such as was experienced at one time at Plainfield, is relatively rare, but demonstrates the possibility of obtaining conditions that will offset the inhibitory effect of low temperatures.

Data are not available for a comparison of temperatures in the sludge compartments of the several tanks under consideration. Such determinations are needed before definite conclusions can be made relative to the effect of temperature in explanation of the differences in tank behavior. There can be no doubt, however, that temperature should be considered in the determination of the required capacity of digestion compartments.

Preparatory Treatment of Sewage

At Schenectady, about 75% of the sewage passes through coarse racks and centrifugal pumps, on the way to the treatment plant, at which all the sewage passes through single racks consisting of 2 by 5/8-in. bars spaced 1½ in. in the clear. No grit-chambers are provided.

At Plainfield, the sewage received only coarse screening at the time of maximum trouble with foaming. Later, a Riensch-Wurl screen having 1/16-in. slots was installed, since which less difficulty with foaming has been experienced.

At Fitchburg, the sewage is screened through coarse racks, with 1¾ in. clear space, and passes through grit-chambers. A small proportion is pumped.

At Rochester, the sewage passes through coarse racks, grit-chambers and Riensch-Wurl screens having slots ⅛ in. in width. During storms, a part passes through a screen having ½-in. slots.

At Rochester, fine screening may account for the freedom from trouble due to foaming and forming of scum; it certainly has been of assistance in these respects at Plainfield. At the latter plant, however, the formation of scum and foam is still excessive.

Obviously, the quantity of organic matter to be digested can be somewhat reduced by fine screening. Gas is more easily liberated from gas-lifted masses of finely divided solids than from masses of coarse solids, which tend to interweave and hold together. Fine screening, therefore, may aid greatly in reducing scum formation. In the light of experience at Fitchburg, however, fine screens cannot be said to be a necessity under all conditions, and the advisability of installing them would appear to depend on the expense involved in disposing of the coarser parts of the suspended solids by fine screening on the one hand, and by tank treatment on the other. There is no doubt that preparatory treatment by fine screening affords a factor of safety in the operation of the tanks.

Kind of Biological Action

As stated previously, biological action increases and decreases greatly with the rise and fall of temperature. The effect of other factors, however, are important, such as adequacy and character of food supply, moisture, oxygen, inorganic poisons, light, antagonistic organisms, waste products of biological activity, and reaction of environment.

Ample food supply and moisture are always present in the sludge compartment of an Imhoff tank. Although evidence is lacking, there is a possibility that different sewages produce sludges of such different composition that corresponding variations are produced in the predominant organisms, with resultant variation in the kind of biological action.

Light, oxygen, and inorganic poisons are of relatively small importance in the processes of digestion. The circulation of liquid between the digestion and the sedimentation chambers, and the periodic withdrawal of sludge largely bring about the removal of the products of decomposition, and it is probable that the "self-poisoning" of the essential organisms in the sludge is not an important factor.

The reaction of the sludge and liquid in the digestion chamber may have a marked effect on the digestion processes. It is difficult to state, however, whether excessive acidity with poor quality of sludge is a cause or a symptom. In certain instances, notably at the Pennypack Creek plant at Philadelphia, Pa., the use of lime is said to have resulted in marked improvement in the behavior of Imhoff tanks. It is also possible that large quantities of carbon dioxide may have a distinct inhibitory effect on biological activity. This seems more significant in the case of scum in which the gases are entrapped, than in sludge from which they can more readily escape.

With scum, however, the substitution of gas for moisture in the voids may render it less easily broken by organic activity than is the case with wet sludge. Furthermore, there is much less opportunity for washing out the final products of biological activity and the correspondingly greater opportunity for self-poisoning of the organisms of decomposition. These conditions are particularly favored by allowing the scum to remain undisturbed for long periods, as was done at Schenectady, during 1922.

No data are available by which the kinds of biological action at the several plants under consideration can be compared, with a view to deter-

mining the causes of the differences in behavior. It seems reasonable to expect a difference between the kind of action in the sludge and that in the scum. It is conceivable that the tendency to form scum may be largely due to the physical characteristics of the tanks and to the degree of comminution of the coarser solids. Any difference in the predominant kind of biological action in the scum, may account for the phenomenon of foaming.

In general, two kinds of decomposition develop: First, the production of carbon dioxide by the splitting of non-nitrogenous organic matters, or fermentation; and, second, the production of methane by the splitting of nitrogenous organic matters, or putrefaction. Both kinds of decomposition may be included in the digestion process.

The initial decomposition of solids in a digestion compartment results in the production of a large proportion of carbon dioxide, due to the fact that fermentation appears to start at a rapid rate. Under favorable conditions, however, the character of decomposition quickly changes and methane becomes the predominating gas produced, roughly, 85% of the total volume. These stages of decomposition appear to be like those observed in the septic tank, as demonstrated by the experiments at Worcester and elsewhere. Unsatisfactory results will be obtained if biological conditions, such as food and environment do not permit the change from carbon-dioxide fermentation to methane decomposition.

It has frequently been suggested that scum and foam formation, which recently at certain plants has become more troublesome than ever before, may be due to acid fermentation of large quantities of mash from the illicit manufacture of alcoholic beverages. There is ample evidence of the presence of large quantities of such materials in sewage and it is probable that they are peculiarly susceptible to fermentation. Whether or not this is an important factor in the formation of gas and scum, such material has been very noticeable in the excessive scum found at some plants.

It may be that certain kinds of action, such as fermentation with production of large quantities of carbon dioxide, do not as rapidly disintegrate, gasify, and render soluble the coarser organic substances as other varieties of action. Therefore, with fermentation, for example, there might be an abnormal accumulation of scum-forming solids tending to overload the sludge compartment or to form gas-lifted masses in such quantities that excessive scum formation is inevitable.

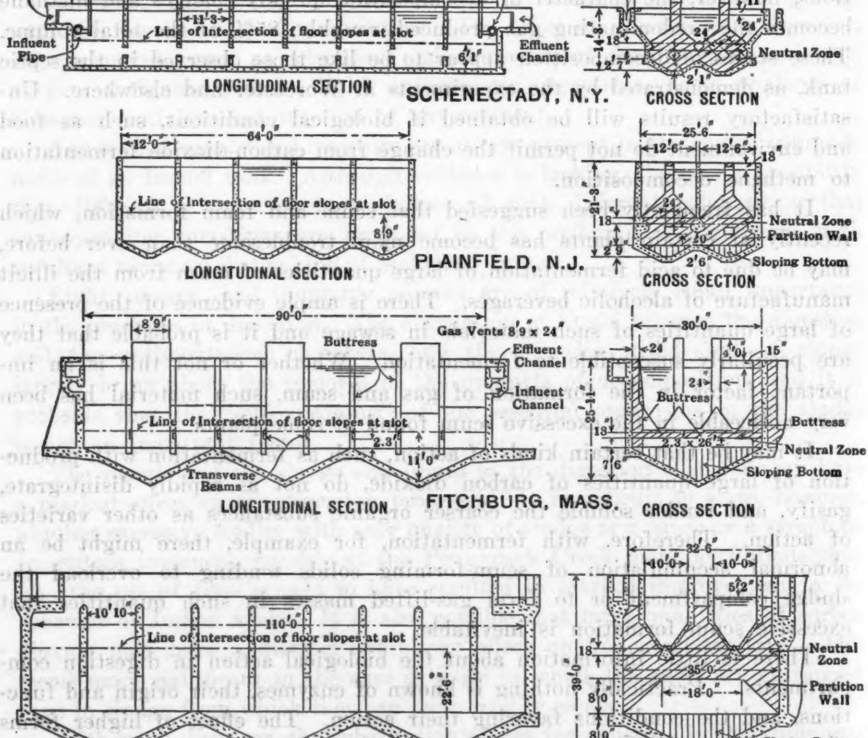
There is little information about the biological action in digestion compartments. Practically nothing is known of enzymes, their origin and functions, and the conditions favoring their action. The effect of higher forms of organisms, possibly antagonistic to the digestive varieties, is also little understood. The agitation of scum for the liberation of gas gives promise of confining the action within the digestion compartment to that which is typical of sludge and thus avoiding difficulty in undesirable biological action which may take place in the more or less compact scum. Although it may be possible to obtain satisfactory digestion in shallow tanks in this manner, the indications are that a much larger sludge compartment

will be required in such tanks to offset the greater volume of sludge and the tendency for scum formation.

Sanitary engineers are looking forward with keen interest for the results of the investigations being conducted jointly by the New Jersey Agricultural Experiment Station, New Jersey State Board of Health, and a committee of two sanitary engineers, a chemist, and a practical operator, all members of the New Jersey Sewage Operators' Association, serving as an Advisory Committee.

TANK DESIGN

The physical characteristics of the tanks are illustrated by Fig. 3. Comparisons are made on the basis of populations served and flows treated during 1922 and not on populations and flows for which tanks were designed.



COMPARISON OF IMHOFF TANKS

FIG. 3.

The tanks differ in length, from 64 ft. at Plainfield, to 110 ft. at Rochester. The fact that most of the solids settle comparatively near the inlet end and in materially decreasing quantities farther inside should be considered in

determining the length of tank. On this account, a comparatively short tank may be more advantageous than a long one.

The digestion space in the tanks at Schenectady is subdivided by partition walls into eight compartments, and that at Plainfield into five compartments. There are no subdivisions of the digestion compartment in the Fitchburg plant, although buttresses are used in the upper part of the side scum compartments. The digestion space of the Rochester plant is subdivided into three compartments by partition walls.

It is important to distribute the sludge throughout the sludge storage space as uniformly as possible, in order that each part of the tank may fully serve its allotted purpose and that no part may be overloaded. During the early days of operation at Plainfield it was found that more than half the sludge was collected in the first hopper, and a large part of the remainder in the second hopper; reversal of flow resulted in a similar distribution of sludge in the hoppers at the other end of the tank. Little or no sludge was collected in the middle hopper.

Reversal of flow in multiple-hopper tanks assists in the distribution of solids. Further aid in equalizing the accumulation of sludge may be obtained by permitting it to flow from one hopper to another by gravity. At Schenectady and at Plainfield, the only opportunity for the sludge to flow from one hopper to another is through the small opening in each of the partition walls. As these openings are only about 2 ft. square (they are smaller at Plainfield) and are some distance above the bottom of the tank, the opportunity for equalization is not good, particularly in the plant at Schenectady in which the sludge must pass through three such openings to reach one of the central compartments.

At Fitchburg, the opportunity for the passage of sludge from one compartment to another is much better, the only interfering members being the transverse beams and side-wall buttresses. The elevation of the beams practically coincides with that of the assumed neutral zone. The most restricted space for the flow of sludge is between the ridge separating the hoppers and the transverse beam, and is approximately 2.3 by 26.0 ft. in vertical cross-section.

At Rochester, a large opening between the sludge compartments affords ample communication.

Distribution of Solids Among Tanks.—It is as important to obtain uniform distribution of solids among the several tanks as within the hoppers of a single tank, but this is difficult to obtain, as has been demonstrated at Schenectady, where it is evident from observation and analyses that one battery of tanks receives sewage containing a much larger quantity of suspended matter and coarser solids than the other. A similar, although less marked, segregation of solids, has occurred at Fitchburg.

Depth of Tanks.—The depths of the tanks below the surface of the sewage, and the distances below and above the bottom of the assumed neutral zone, are given in Table 9.

An essential difference between the good and bad-acting tanks is in their respective depths. The Schenectady and Plainfield tanks are much shallower

than those at Fitchburg and Rochester. The influence of depth of tank on the action within it is not clear. One theory is, that in the deeper tanks there is a longer path through which the gas-lifted solids must ascend in order to form an accumulation of scum at the top,—and that this prolonged travel affords opportunity for the escape of the gases after which the solids may return to the bottom, thus avoiding their accumulation as scum.

TABLE 9.—DEPTHS OF TANKS AND COMPARTMENTS.

Plant.	Maximum depth, in feet, below surface of sewage.	Maximum distance, in feet, below bottom of neutral zone.	Maximum distance, in feet, above bottom of neutral zone.
Schenectady	13.8	4.6	9.2
Plainfield	19.8	7.25	12.55
Fitchburg	24.7	9.5	15.2
Rochester	33.9	21.0	12.9

When the sludge has accumulated to the level of the bottom of the neutral zone, the maximum possible length of travel is greater at Fitchburg and Rochester than at Schenectady. At Plainfield and Rochester, the lengths of travel are practically the same. It is perhaps more significant that during most of the year, when the accumulation of sludge should be considerably below the bottom of the neutral zone, particularly at Fitchburg and Rochester, the length of travel at these plants is much greater than that at Schenectady and Plainfield.

In the deeper tanks, the gas is generated under greater pressure and the bubbles entrained in the sludge are subject to considerable expansion on rising, thus possibly aiding in their escape from the solids; likewise, there is a deeper accumulation of sludge, which tends toward greater density.

The maximum possible depths of sludge at Fitchburg and at Rochester are approximately twice and four times, respectively, as great as that at Schenectady.

The greater depths of the tanks at Fitchburg and Rochester appear to explain in part the absence of excessive scum formation and the comparatively greater density of sludge at these two plants.

Sludge Compartments.—Dr. Imhoff's allowances for capacity of sludge compartments for separate and combined sewages are 1.2 and 1.8 cu. ft. per capita.* This allowance of 50% extra capacity for combined sewage is somewhat in excess of the extra quantity of solids in the sewage of Fitchburg and Rochester, which was 36%, but is probably necessary because of the greater proportion of undigestible solids in the combined sewage. It appears that for the present population, the tanks have capacities in excess of those recommended by Dr. Imhoff (Table 10). The compartments at Rochester are 43% in excess of his allowance.

Dr. Imhoff has pointed out the necessity of having a larger sludge compartment for a shallow than for a deep tank. It has been shown that the sludge drawn at Schenectady and Plainfield contains only about 7½% solids,

* *Engineering News*, Vol. 75 (1916).

or one-half the proportion in that drawn at Fitchburg and Rochester. The volume of sludge at Schenectady and Plainfield, therefore, would be twice that at Fitchburg and Rochester for an equal weight of solids. If it is reasonable to assume that the Schenectady and Plainfield sewages would produce a sludge containing 15% solids in deep tanks, such as those at Fitchburg and Rochester, the allowance for volume of sludge to be stored in the Schenectady and Plainfield tanks as built should be doubled on account of their shallowness. The sludge capacities provided at Schenectady and Plainfield are only slightly more than one-half that required by this reasoning.

TABLE 10.—CAPACITY OF SLUDGE COMPARTMENTS.

Plant. (1)	Dr. Imhoff's recommended allowance, in cubic feet per capita. (2)	Dr. Imhoff's allowance modified because of shallowness of tanks, in cubic feet per capita. (3)	Actual gross capacities, in cubic feet per capita. (4)	Actual net capacities after deducting neutral zone, in cubic feet per capita. (5)
Schenectady............	1.2	2.4	1.40	1.02
Plainfield............	1.2	2.4	1.49	1.22
Fitchburg............	1.8	1.8	1.88	1.47
Rochester............	1.8	1.8	2.49	2.21

Although it has been found to be convenient to consider the capacity of sludge compartments in terms of gross capacity, it is important to recognize that it is not wise to depend on utilizing the entire compartment for the storage of sludge.

Doubtless, the depth of the neutral zone should be determined from a consideration of several conditions, such as the depth of the sludge accumulation and proportion of uncomminuted solids. Probably it should be deeper in shallow tanks treating fresh unscreened sewage than in deep tanks treating stale screened sewage. In this study, however, it seems reasonable to allow the same depths in all cases, namely, 18 in. In order to arrive at the proper net available storage capacity, the effect of deducting the volume of the neutral zone from that of the gross compartments, depends on the design of the tank. Thus, making this deduction, the gross capacities are reduced approximately 11% at Rochester and 27% at Schenectady.

The load on the digestion compartment may be expressed in pounds of suspended solids deposited per year per cubic foot of its capacity. A comparison of the loads computed in this manner, based either on the gross or the net capacity of the sludge compartment (Table 11), shows that the load at Rochester is the smallest and that at Fitchburg the greatest. At Schenectady and Plainfield, the loads are approximately 1½ and 2½ times as great as that at Rochester, if deduction is made for the neutral zone.

These loads are based on the quantity of solids deposited without any reference to their partial digestion or to the volume of sludge formed by them. As the solids lie in the tank, they are partly digested and allowance should be made therefor. During the spring, at times of maximum load, it is

not likely that more than one-half the digestion has taken place. Assuming that digestion has progressed that far, 20% of the solids in the separate sewage and 15% in the combined sewage, have disappeared leaving 80% and 85%, respectively. If allowance is made for digestion and, if the large volume of thin sludge at Schenectady and Plainfield proportionately limits the load which can be placed on the sludge compartments, the relative loads by volume on the sludge compartments at the various plants, calling that at Rochester unity, are shown in Table 12.

TABLE 11.—LOADS UPON SLUDGE COMPARTMENTS

Plant. (1)	Suspended solids based on gross sludge capacity, in pounds per year per cubic foot. (2)	Ratio to load at Rochester. (3)	Suspended solids based on net sludge capacity, in pounds per year per cubic foot. (4)	Ratio to load at Rochester. (5)
Schenectady....	21.9	1.94	30.0	2.44
Plainfield......	15.5	1.37	18.9	1.54
Fitchburg......	24.1*	2.13	30.9*	2.51
Rochester......	11.3	1.00	12.3	1.00

* Includes secondary tank sludge pumped to Imhoff tanks.

It appears that the load at Schenectady is 4.6 times as great per cubic foot as that at Rochester. If the load on the tanks at Rochester is assumed to be correct, the Schenectady tanks are greatly overloaded.

TABLE 12.—RELATIVE LOAD BY VOLUME BASED ON ACTUAL NET VOLUME OF SLUDGE COMPARTMENTS.

Plant.	Relative load by volume.
Schenectady..................................	$\frac{30.0 \times 0.80 \times 2}{12.3 \times 0.85} = 4.6$
Plainfield..................................	$\frac{18.9 \times 0.80 \times 2}{12.3 \times 0.85} = 2.9$
Fitchburg..................................	$\frac{30.9 \times 0.85 \times 1}{12.3 \times 0.85} = 2.5$
Rochester..................................	$\frac{12.3 \times 0.85 \times 1}{12.3 \times 0.85} = 1.0$

Of course, the Rochester sludge compartment may be too large, in which case, the relative loads by volume would make the compartments at Schenectady and Plainfield appear to be more inadequate than they actually are. If the relative load by volume at Fitchburg is assumed as 1, the loads by volume at Schenectady and Plainfield become 1.8 and 1.2, respectively, and that at Rochester becomes 0.4. It is not the purpose in this paper, however, to consider the proper load by volume, but rather to compare existing conditions at the several plants in order to arrive at an explanation of the differences in behavior of tanks.

Although these relative loads by volume may be based on a rather uncertain combination of assumptions, they are, nevertheless, significant. The

larger loads at Schenectady and Plainfield, as compared with those at Rochester and Fitchburg, may explain in part the difficulties of operation at the former places. If the load on the sludge compartment is too great, the sludge must either be drawn before it has become thoroughly digested and is, therefore, offensive, or be allowed to accumulate, thus encroaching on the neutral zone, or both. The latter condition is particularly favorable to scum formation.

Scum Compartments.—The largest per capita volume of scum compartments is at Fitchburg, those at Schenectady and Plainfield being only about half as large.

TABLE 13.—SCUM COMPARTMENTS.

Plant.	Volume, in cubic feet per capita.	Loading, in pounds per year per cubic foot.
Schenectady	0.76	40.4
Plainfield	0.72	32.1
Fitchburg	1.59	28.5
Rochester	0.55	49.7

If the tanks were functioning under ideal conditions, that is, with gas entirely liberated as the solids rise, and all the solids in the form of sludge, there would be little logic in considering the load on the scum compartments. Under existing conditions, however, with such large accumulations of scum, the quantity of solids tributary to the scum compartments should be considered. For the sake of comparison, the loading has been computed on the basis of the entire weight of solids deposited in the tanks, no effort being made to subdivide the solids into those parts which may constitute sludge and scum, respectively (Table 13). The smallest load is found to be at Fitchburg, and the greatest at Rochester. At Rochester, however, the coarse solids have been removed by screening, which may account for the small accumulation of scum, notwithstanding the apparent heavy load. The excess of load at Schenectady, over that at Fitchburg, indicates one reason for the trouble due to scum at the former plant.

The load on the scum compartments at Plainfield is somewhat greater than that at Fitchburg. Taking into consideration the shallowness of the Plainfield tanks—a condition favorable to scum formation—the load may be great enough to account in part for the trouble due to scum, notwithstanding the screening of the Plainfield sewage.

Some engineers have maintained that small gas-vent area is advantageous, whereas others have argued that the area should be large. Although the areas of the gas vents are substantially equal in all the plants, those at Rochester comprise a much larger proportion of the total tank area than the others. This does not tell the whole story, however. A more logical method of studying the effect of gas-vent areas would seem to be to compare them with the volume of tributary sludge space, or better yet, with the loading in pounds of sludge solids, as given in Table 14.

TABLE 14.—GAS-VENT AREAS.

Plant.	Area of gas vents, percentage of total tank area.	Area of gas vents, in square feet per cubic foot of sludge space.	Loading, in pounds of solids per year per square foot of gas-vent area.
Schenectady	14.8	0.034	644
Plainfield	14.3	0.023	674
Fitchburg	15.0	0.031	777 { side vents 335; central vents 2040 }
Rochester	26.8	0.0133	849

On these bases of comparison, the gas vents at Rochester are much less liberal than the others.

The load of solids and probably, therefore, the volume of gas tributary to the vents, is much greater at Fitchburg and Rochester than at Schenectady and Plainfield. The absence of trouble at the former plants tends to support the claim that small vents are preferable, although the differences in loads cited are not very great. At Fitchburg, however, there is a marked difference between the equivalent area of the side vents and of the central vents, the latter being nearly six times as heavily loaded as the former. Experience at this plant does not indicate a great difference in behavior between the side and central vents. There has been slightly more foaming in the latter. The evidence furnished by the plants under consideration, is not sufficient to justify a conclusion in regard to the comparative merits of the large and the small gas-vent areas.

The theory has been advanced that gas-lifted sludge hitting against the inclined bottom of the sedimentation compartment may thus lose, through agitation, a part of its gas. Thus, a design like that at Schenectady, which makes most of the sludge directly tributary to the scum compartment, may be less advantageous than the Fitchburg design in which a larger proportion of the gas-lifted sludge must roll or slide a considerable distance on the overlying concrete.

Another theory is that gas-lifted sludge held in the inverted V-trough beneath the sedimentation compartment, as in the central scum space at Fitchburg, keeps the solids wet and permits more active digestion than in the gas vents where a greater proportion of the gas may escape more quickly and a compact mass of scum be formed, as at Schenectady.

OPERATION

Imhoff tanks should be operated in such manner as best to develop their two important functions: The removal from sewage of suspended solids and their digestion to a sufficient degree to render them inoffensive when spread on drying beds and used for filling or fertilizer, or finally disposed of otherwise.

Efficiency of sedimentation should be controlled as far as practicable with the aid of determinations of the proportion of suspended solids removed from the sewage during its passage through the sedimentation compartment. It should be recognized that digestion of organic matter results from biological

action, and that operating conditions should be made as favorable as possible to such action.

All the solids entering the digestion compartment must be digested or removed in the form of sludge or scum. It is of the utmost importance for the operator to know from time to time to what extent solids have accumulated in the digestion compartment in order that he may compare the quantity of such accumulation with the capacity provided for it. In some instances, this can be determined by the measurement and analysis of sludge and scum, but, in others, this has proved impracticable, if not impossible. Where this method is impracticable, helpful information can be obtained from analyses of tank influent and effluent and measurement of volume of sewage treated, together with data relating to sludge drawn.

The operator should carefully study the condition of sludge available for drawing, particularly during the early spring, in order to determine the degree of digestion that has taken place, and govern the drawing of sludge accordingly, giving due consideration to the relation between the volume of sludge on hand and the capacity of the digestion chamber. The drawing of sludge in the spring should be postponed as long as practicable in order to reduce the danger of removing offensive smelling sludge. On the other hand, it must not be allowed to accumulate to the extent of causing foaming or other difficulty.

Intelligent and skillful plant performance is greatly aided by certain analytical records which enable the operator to know what is being accomplished by the tanks and how their efficiency can be increased. The volume of sewage should be accurately measured. Estimating the tributary population from year to year will enable the operator to compile some of his analytical and load data on a per capita basis, which is very helpful in some instances. The sludge and scum removed from the tanks should be accurately measured and the solids determined. It is also desirable to determine the proportion of organic matter in the sludge and scum removed. From measurements and analyses, the proportion of the solids which disappear through digestion should be computed. Determinations of the temperature in the sludge compartment will aid in understanding the conditions within the compartment.

Satisfactory results in the operation of such tanks are as dependent on intelligence and skill in operation as on good design. All the plants under consideration have had technical supervision and the differences in results obtained cannot be attributed to lack of skillful operation.

Conclusions

This study of the available evidence afforded by the Imhoff tanks at these four plants, summarized in Table 15, leads to the following conclusions:

1.—The mineral and heavy relatively stable organic matter of the combined sewage at Fitchburg and Rochester may tend to prevent the formation of excessive scum and foam by weighting down the sludge.

2.—Coarse and uncomminuted solids at Schenectady are probably an important factor in the excessive formation of scum.

TABLE 15.—SUMMARY OF ESSENTIAL DATA ON IMHOFF TANKS.

Item.	Schenectady, N. Y.	Plainfield, N. J.	Fitchburg, Mass.	Rochester, N. Y.
Tributary population (1922)	65 000	40 000	38 000	260 000
Character of Sewage:				
Flow, in million gallons per day	6	3.4	3.4	32
Separate or combined sewers	Separate	Separate	Combined	Combined
Strength (suspended solids), in parts per million	163	166	219	163
Freshness	Fresh uncomminuted	Stale	Fresh	Stale
Industrial wastes	Practically none	Practically none	Small amount	Small amount
Hardness of water supply, in parts per million	130	88	10	65
Temperature, in degrees, Fahrenheit	Av. (1922) 56.1 Max. 64.7	46–70	No data	Av. 54
Preliminary Treatment:				
Screening	Coarse rack	Fine screens, 1/16-in. slot	Coarse racks	Fine screens, 1/8-in. slots
Grit-chambers	None	None	Yes	Yes
Imhoff Tanks:				
Sedimentation period, in hours	3.3	3.4	6.4	1.1
Depth of Tanks:				
Total maximum water depth	13 ft. 9 in.	19 ft. 9 in.	24 ft. 8 in.	33 ft. 10 in.
Sludge Compartment:				
Depth below plane of slots	6 ft. 1 in.	8 ft. 9 in.	11 ft. 0 in.	22 ft. 6 in.
Depth below 18-in. neutral zone	4 ft. 7 in.	7 ft. 3 in.	9 ft. 6 in.	21 ft. 0 in.
Distance of lowest point of overflow below 18-in. neutral zone	2 ft. 6 in.	2 ft. 0 in.	2 ft. 0 in.	13 ft. 0 in.
Depth below overflow to adjacent compartment	2 ft. 1 in.	5 ft. 3 in.	7 ft. 6 in.	8 ft. 0 in.
Cubic feet per capita	1.40	1.49	1.88	2.40
Number of hoppers	8	5	3	3
Ease of intercommunication	Poor (2-ft. square opening)	Poor (opening 20 by 24 in.)	Good	Good
Scum Compartment:				
Cubic feet per capita	0.76	0.72	1.59	0.55
Area, gas vents; percentage of tank area	14.8	14.3	15	26.8
Area, gas vents; square feet per cubic foot sludge capacity	0.034	0.023	0.031	0.0133
Loading, Deposited Solids:				
Pounds per year per cubic foot sludge and scum space	14.2	10.4	13.1	9.2
Pounds per year per cubic foot sludge space	21.9	15.5	24.1	11.3
Pounds per year per cubic foot sludge space after deducting 18-in. neutral zone	30.0	18.9	30.9	12.3
Pounds per year per cubic foot scum space	40.4	32.1	28.5	49.7
Pounds per year per square foot gas-vent area	644	674	777	849

3.—If the water supply is hard, the insoluble soaps formed, constitute a substantial increment in the suspended solids of the sewage and may favor the formation of foam and scum.

4.—The variation in the quantity of suspended solids to be removed from different sewages is so great that the design of the digestion compartment should be based on the quantity of solids to be deposited in it, rather than on a general assumption of a definite number of cubic feet per capita.

5.—Temperature is a factor of fundamental importance in the digestion process.

6.—The required capacity of the digestion compartment is governed largely by the available temperature and by the duration of the period of low temperature.

7.—Difficulties will be minimized by drawing sludge as early in the spring as inoffensive material can be obtained, by continuing the drawing at a rate sufficient to provide as small an accumulation in the sludge compartment as practicable during hot weather, and by removing before cold weather, all sludge except that required for seeding.

8.—Fine screening by reducing the quantity of digestible solids to be deposited from the sewage, reduces the load on the digestion compartment correspondingly, and by removing the coarser matter particularly favorable for scum formation tends to reduce difficulty from that source.

9.—Fine screening appears to have been beneficial at Plainfield and Rochester; although not a necessity, it affords a factor of safety in the operation of Imhoff tanks.

10.—The advisability of installing fine screens appears to depend on the relative cost of disposing of the coarser part of the suspended solids by fine screening on the one hand, and by tank treatment on the other.

11.—It is important to distribute the deposited solids as uniformly as possible throughout the digestion compartment; for this reason a relatively short tank is advantageous.

12.—Digestion compartments should be subdivided as little as practicable, and liberal opportunity should be afforded the sludge to spread uniformly from one end of the tank to the other.

13.—Lack of uniform distribution of sludge throughout the digestion compartment may have been an important factor in the unfavorable action at Schenectady and Plainfield.

14.—Frequent reversal of flow is necessary for successful operation of multiple-compartment tanks.

15.—It is important to obtain nearly equal distribution of solids among the several tanks; failure to accomplish this at Schenectady has been a factor in the difficulty of operation.

16.—There appears to be a decided advantage in the greater depth of tanks at Fitchburg and Rochester in preventing excessive scum formation and in providing sludge with a comparatively large proportion of solids.

17.—In the design of the digestion compartment consideration should be given to the probable density and corresponding volume of the sludge as it will lie in the tank.

18.—If tanks must be shallow, substantial additional capacity must be provided in the digestion compartments.

19.—Taking into account both the load in pounds of deposited solids and the probable density of the sludge, the digestion compartments at Schenectady and Plainfield appear to be relatively much smaller than those at Fitchburg and Rochester, a condition offering one explanation of the difficulties at the former plants.

20.—Evidence is so meager and conflicting that a final conclusion regarding the comparative merits of a small and a large proportion of gas-vent area is not justified.

21.—Any opportunity for the breaking of rising gas-lifted sludge, as by impact on the floor partition between the sedimentation and digestion compartments, may be of advantage.

22.—Successful operating results depend as much on intelligence and skill in operation, as on correct design.

23.—Complete accurate operating data are highly desirable, and very helpful to intelligent and skillful operation.

24.—Little is known about the kind of organisms, or the biological action in Imhoff tanks. The absence of such information has made it necessary to confine this study to the structural, physical, and chemical differences. They appear to explain many results without consideration of biological action. The latter may have been a factor, possibly as a result of certain structural features, rather than as a primary cause of unfavorable action.

25.—The differences in results obtained are not explained by a single condition, but appear to be due to many factors which in the aggregate make a wide difference between the two pairs of plants. The essentially unfavorable conditions at Schenectady and Plainfield are:

(*a*) Shallowness of tanks;
(*b*) Inadequate digestion compartments;
(*c*) Impossibility of uniform distribution of sludge throughout digestion compartments;
(*d*) Large number of digestion compartments, making it impracticable to determine volume and density of sludge in them and difficult to cope with sludge and scum problems;
(*e*) Absence of heavy solids from street washings;
(*f*) Relatively large proportions of insoluble soaps.

Need for Research

Additional information is much needed on both the biological and physical phases of the process of digestion, as to the:

1.—Temperatures which prevail in digestion compartments throughout the year in different parts of the country, preferably determined by recording thermometers which should be placed both in scum and sludge compartments.

2.—The reaction, or hydrogen-ion concentration of sewage, liquid in digestion compartment, sludge, and scum.

3.—The rate of digestion at different temperatures, when all other conditions are controlled so that they are as nearly uniform as possible, in the parallel tests.

4.—Variation in the volume of gases evolved at different temperatures, with the object of utilizing such determinations as an index of the rate of digestion.

5.—Variation in the composition of gases evolved at different plants and temperatures, with a view to utilizing the composition of the gas as an index of the character of decomposition in progress.

6.—Kind and functions of the organisms predominating in sludge and scum compartments, respectively, under varying conditions such as different reactions and temperatures.

7.—The effect of different depths, by operating tanks in exact parallel in all other respects including the loads.

8.—The effect on permissible load, of removing the coarse solids from the sewage before it is introduced into the tanks, determined by tests in exact parallel in all other respects.

9.—The effect on permissible load, of continuous artificial agitation of scum to an extent which will cause the solids to remain in the sludge mass practically all the time.

Many other tests might be suggested, but those enumerated appear to be particularly promising. In order that results may be as conclusive and valuable as possible, it is of the utmost importance that, in parallel tests, the variables be reduced, preferably to a single condition, such as temperature, depth, or removal of the coarser solids.

COMPARATIVE TESTS
ON EXPERIMENTAL DRAFT-TUBES
Discussion*

BY MESSRS. JOHN R. FREEMAN, CLEMENS HERSCHEL, F. W. SCHEIDENHELM, AND RICHARD A. HALE.

JOHN R. FREEMAN,† PAST-PRESIDENT, AM. SOC. C. E.—It is said that there is nothing new under the sun. About 50 years ago, the speaker saw a turbine outlet that took cognizance of the principles of the "Hydraucone". This was on a Swain turbine. Richard A. Hale, M. Am. Soc. C. E., assisted on a test of such a turbine about 53 years ago, and will clearly recall the shape of the outlet passage, which gave evidence that the inventor appreciated the principle involved in the design of the "Hydraucone".

The evolution of the American turbine during the 80 years following the inventions of that remarkable hydraulician, Uriah A. Boyden, of Lowell, Mass., discloses a reliance more on plain common sense and the gift of visual imagination than on algebraic research and formulas.

A fundamental principle of hydraulic design is to avoid all abrupt changes of flow, which cause eddies and turmoil, and thereby waste of energy. The great engineer and artist, Leonardo di Vinci, nearly 500 years ago, showed that he appreciated this principle better than some comparatively recent designers.

The path of a particle of water through a modern inward and downward flow turbine and its draft-tube is such a complex of spiral and helix, with changing curvature, that a description of its course by an algebraic formula is hopeless. Nevertheless, a designer with common sense and good visual imagination can trace the pathway of the fluid filaments as he works over a model, so as to avoid abrupt changes in direction and the creation of eddies.

The late James B. Francis, Past-President, Am. Soc. C. E., and Mr. J. C. Hoadley worked out on the drafting-board the hypothetical pathway of a particle of water through the runner of a Boyden turbine. The late Professor De Volson Wood devoted most of one summer vacation to developing some wonderful equations of flow for that type of runner in which the motion was all in one plane, as in the Boyden wheel. After Francis had pointed the way to greater compactness of design and to the creation of turbine units of much higher power, by his inward and downward flow designs, the pathway of a particle of water through the turbine became too complicated for following precisely on a drafting-board, but not too complicated for following in imagination, in a general way, using a model.

---

* Discussion of the paper by C. M. Allen, M. Am. Soc. C. E., and I. A. Winter, Esq., continued from April, 1924, *Proceedings*.

† Cons. Hydr. Engr., Providence, R. I.

Thus, it happened that the next great advances were made by men without knowledge of algebra or calculus, but strong and clear in visual imagination, and possessed of the idea of leading water around easy curves, with carefully guided expansion and the avoidance of eddies. The first of these was Asa M. Swain, a skilled pattern-maker, and the second was John B. McCormick, a country blacksmith, creator of the "Hercules" turbine.

The idea of recovering velocity head from the turbine efflux, as in the modern draft-tube, seems to have originated about 75 years ago with Uriah A. Boyden, in many ways a remarkable, self-trained engineer, with a wonderfully clear head.

The Boyden turbine runner being of the outward flow type, his "diffuser", or head regainer, could not be applied in the form of an expanding cylindrical draft-tube, but was constrained to take the shape of an expanding area between horizontal, curving disks; and by the application of these diffuser disks to some of the ancient turbines in Lowell, he materially increased their percentage of useful effect. The speaker first saw them fifty years ago on the old turbines of the Atlantic Mills at Lawrence, Mass., installed in 1847, and reputed to have given nearly 90% efficiency at full gate.

Francis was the first experimental physicist or engineer to measure accurately the remarkable effects obtainable in recovering velocity head by means of gently expanding the cross-section of the conduit; but he did not extend his experiments to cover all the possibilities of the expanding turbine draft-tube, with its twisting or spiral currents, which aid the diffusion and increase the effective angle of divergence far beyond the 11° which he found to be most efficient.

When the proposed National Hydraulic Laboratory is established, some instructive experiments can be made for measuring the efficiency of various angles of diffusion with various amounts of twists. At present, there is abundant proof that high angles of divergence are not inconsistent with economy.

A short draft-tube and a wide angle of diffusion is compelled by the practical necessities of almost every installation. The small angle of 11° of the Francis Venturi tube is utterly out of the question. This short draft-tube is required, first, because the practical working limits of atmospheric suction will not permit a greater vertical height than 25 ft., and, second, by the extravagant cost of using a deep pit with a subsequent ascent to the tail-race.

Under these limitations of short draft-tube and wide angle, the whirling current with its centrifugal force is an aid. If the fluid filaments were moving parallel to the axis and without this twist, the current would tend to leave the inside of these wide angled draft-tubes and to shoot out in a jet, slightly divergent, leaving the space between this jet and the walls of the draft-tube filled with water in turmoil, thus destroying energy instead of conserving it by a diffusion and gradual reduction of velocity.

The hydraucone is simply a common-sense device for lessening the tendency to form eddies in a short, widely divergent draft-tube, and for gently changing the vertical motion of descent into a horizontal motion outward to the tail-race with the smallest possible amount of turmoil.

The gently rounding flare of the internal cone embodies the same idea that was evolved by Swain in his turbine setting 53 years ago. The modern designer, aided by mathematics, gives a more scientific shape to the vertical section of the passageway, doubtless conserving the energy more than the Swain design, in which expansion probably was too rapid to permit the current to follow it without more or less eddying.

It is earnestly hoped that some investigator will experiment on a series of models of draft-tubes with a wide cone base, having different angles of divergency and provided with a device by which various degrees of twist can be given the current.

An excellent means for measuring this twist consists of a modification of the Pitot tube devised by the late Hiram F. Mills, Hon. M. Am. Soc. C. E., for measuring velocities in factory penstocks at Lawrence. This instrument is composed of a brass tube, 1 in. in diameter, containing about twenty interior tubes, each about $\frac{1}{8}$ in. in outside diameter and having a $\frac{1}{16}$-in. hole. These internal tubes were all so shaped and placed that their ends were brought out through holes in the main tube at various distances on a straight line and finished smooth and flush with the surface. Curiously, it was found that the flush orifice gave just as good an indication of velocity, when pointed up stream as the common adjutage or nozzle of the ordinary Pitot tube.

When this composite tube was inserted through a stuffing-box in a mill penstock, the differences in velocities from center to circumference produced different pressures at the several orifices, as measured by a series of glass piezometer tubes on a manifold scale-board to which the internal tubes were severally connected. The apparatus was fitted with a graduated circle. By revolving it, two positions were found for any tube, giving pressure agreeing with that shown by the outside piezometers. The point midway on this dial between these two positions would indicate the direction of current. From observations on each of the small orifices, the angle of whirl at various points from center to circumference could be easily determined. Using this instrument at various factory penstocks, the speaker has found a surprisingly large amount of whirl even in a long penstock, if it had an unsymmetrical approach or contained an angle.

The possible effect of air bubbles must be carefully considered in draft-tube design, particularly in those cases where the tube is not vertical. The speaker has been much impressed by the remarkably large quantity of air bubbles sometimes appearing in the wheel-pit around the outlet of a draft-tube. In one of the most notable cases, at first it seemed that these proved the existence of a leak in the draft-tube; but after careful examination no such leak could be discovered and a closer study indicated that they came from the release of air previously dissolved in the water and shaken out by the violent agitation during the passage through the turbine, or discharged by the sudden release of pressure during the passage between the turbine guide and the top of the draft-tube, very much as bubbles of gas are released when soda water is drawn.

In a well designed vertical draft-tube, this air, released at the top of the tube, is carried along and discharged without trouble, however great its quan-

tity, just as the air bubbles are carried along in the Frizell or Taylor type of hydraulic column air-compressor; but in draft-tubes like many of those built 30 years ago, at an angle of 45° with the vertical, there are possibilities that large bubbles of air will collect on the upper side of the draft-tube, below the angle, where it changes from the vertical to the slope, and that these may seriously interfere with the regulation of the wheel by creating pulsations, which, at times, may cause a serious loss of vacuum.

CLEMENS HERSCHEL,* PAST-PRESIDENT, AM. SOC. C. E.—This discussion has already brought out how much more need there is for further experiments, which reminds the speaker of one of his pet subjects, that is, an endowed hydraulic laboratory.

Many engineers who have made experiments know how heartrending it is to break off just when one is on the verge of completing an investigation. Yet, it is constantly done. Most hydraulic investigations are incomplete for that very reason; they stop too soon. The kind of hydraulic observatory or laboratory, as one may chose to call it, that is contemplated, may be compared to an astronomical observatory, which is an endowed institution and of no "earthly" use, as might be said, to put it bluntly, but is heavily endowed, and consequently is enabled to constantly reach out farther and farther into the universe.

If there were in hydraulic investigations one mere approximation to what is thus done in astronomy, all engineers certainly would be happier. This is an appeal to all hydraulicians to support this project in any and every way they can and whenever it comes to their notice. Already there is a society for this purpose in France; it has been mentioned here—the Hydrotechnic Society of Grenoble. Its leader—M. Augustin Blanchet—is well known to the speaker. Some day, perhaps very soon, these efforts to create such an endowed hydraulic observatory or laboratory, will crystallize in the United States.

Two apt quotations may illustrate the need of an endowed hydraulic observatory, and of placing it in charge of engineers who will make its operation their life work. Said Castelli, the hydraulician, in 1628: "* * * these knowledges, though of things next our senses, are sometimes more abstruse and hidden than the knowledge of things more remote; and much better, and with greater exquisiteness, are known the motions of the Planets, and Periods of the stars, than those of Rivers and Seas; * * *." And Herbert Spencer (1820–1903) supplements this, when he says: "The value of an experiment depends on the skill of the experimenter".

F. W. SCHEIDENHELM,† M. AM. SOC. C. E.—As an onlooker in this battle of the draft-tubes, the speaker desires to touch on one phase which affects the interpretation of not merely the tests which are the subject of the paper, but also other tests of draft-tubes. This phase concerns the relative and absolute accuracies of the test results. The speaker understands that the authors believe these results to be reliable within about 0.2 of 1%, as far as efficiencies are concerned.

* Hydr. Engr., New York, N. Y.

† Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

Referring to Fig. 21,* the two points of Series S_2-H-9 showing the highest efficiencies indicate a difference of approximately 1 per cent. They do not correspond to exactly the same power, but, apparently, if they were corrected to equal power, there would be a discrepancy of even somewhat more than 1 per cent. Again, in Fig. 22,† if the two highest efficiencies of Series S_2-J-2 were corrected to the same power, there would seem to remain a discrepancy of approximately 0.6 of 1 per cent.

In Fig. 25,‡ Series S_2-M-5 is noted as a check on Series S_2-M-4; therefore, on the repetition of the test, one would expect the two test results at about 0.228 h.p. to correspond. Apparently, however, there is a discrepancy of about 0.7 of 1 per cent.

Some of these discrepancies may be due to errors of observation or in reduction of the data, others to errors in the preparation of the diagrams for this valuable paper, but, in general, they seem to be due to outright differences in actual performance. The speaker believes it would be interesting if the authors were to give their views on the question as to whether the nature of such discrepancies warrants taking them into account and whether, in general, the discrepancies are not sufficiently important to vitiate the general conclusion that the results are correct to within about 0.2 of 1 per cent.

Where there is difference in maximum power, the curves must be either shrunk or enlarged, so to speak, in order to obtain the same proportionate power, but the discrepancies in point appear not to be due to differences in power. In the first two cases (Figs. 21 and 22), the points compared were obtained during the same test runs and, therefore, refer to the same maximum power. Thus, as to Fig. 21, the two points of highest efficiency shown for Series S_2-H-9 are represented by small circles. If one were to draw a curve taking into account only the higher of the two highest efficiencies for the series, the curve would be quite different from that actually drawn. Either one of the two points could be included in a curve passing through all the Series H-9 points, except only the other of the two points of highest efficiency.

The speaker realizes that, in turbine tests with certain types of draft-tubes, a critical condition sometimes is found near the point of maximum efficiency, that is, the combined turbine and draft-tube performance is probably affected by and sensitive to eddies, presumably within the draft-tube. (The region of such instability is not necessarily at maximum efficiency; it may be in the region of a power greater than that at which maximum efficiency is obtained.) In the present instances, the points on both sides of the immediate region of maximum efficiency are consistent with each other; in fact, one could ask for nothing better.

However, it must be recognized that maximum efficiency is an item of great, and usually the greatest, importance. If, then, one draws a curve splitting the differences, so to speak, between two such peak points as have been mentioned in the cases of Figs. 21 and 22, is it safe to assume that the resulting power-efficiency curve involves at maximum efficiency the same

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1834.

† *Loc. cit.*, p. 1835.

‡ *Loc. cit.*, p. 1837.

accuracy, say, within 0.2 of 1%, as those parts of the curve outside the regions of maximum efficiency?

RICHARD A. HALE,* M. AM. SOC. C. E.—The speaker had his first experience in the testing of water-wheels in June, 1869, at Lowell, Mass., as an assistant to the late Hiram F. Mills, Hon. M. Am. Soc. C. E. Mr. Mills made tests of water-wheels of various types and patterns.

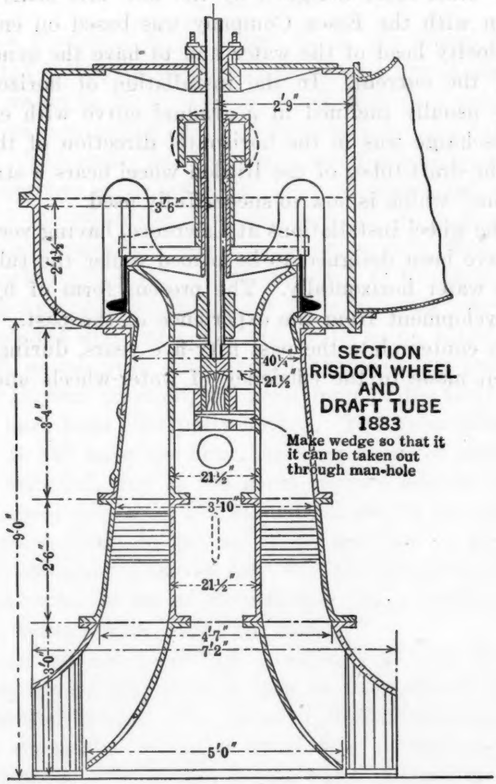


FIG. 40.

Of these, the best efficiency—between 82 and 83%—was given by the Swain wheel, invented by Asa M. Swain, of North Chelmsford, Mass. The bottom casting of the frame supporting the guides and regulating gate of the Swain wheel was made of a flaring conical shape, in order to change the flow of the water discharged to a horizontal direction with as little disturbance as possible. This arrangement doubtless assisted in developing a higher efficiency in the wheel.

In 1883, the Risdon Water Wheel Company installed a wheel, 40 in. in diameter, with a diffuser attached,† in the Lawrence Woolen Company's Mill

* Engr., Essex Co.; Cons. Hydr. Engr., Lawrence, Mass.

† "The American Mixed-Flow Turbine and Its Setting", by Arthur T. Safford, M. Am. Soc. C. E., and Edward Pierce Hamilton, Jun. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Fig. 34, p. 1289.

at Lawrence, Mass. (Fig. 40). The vertical draft-tube had a total length of 9 ft. and enlarged gradually from the bucket outlets to the bottom of the tube. The central part of the draft-tube was of conical shape, forming an annular ring through which the water was discharged in a horizontal direction in the race. This design was not developed in other wheel installations for some unknown reason.

The form of draft-tubes designed by the late Mr. Mills during his fifty years' connection with the Essex Company was based on enlarging the area to regain the velocity head of the water and to have the general discharge in the direction of the current. In the installation of horizontal wheels, the draft-tubes were usually inclined in a gradual curve with enlarging area so that the final discharge was in the horizontal direction of the flowing water. The setting of the draft-tubes of the Risdon wheel bears a strong resemblance to the "hydraucone" which is now so successfully used.

In many of the wheel installations at Lawrence, having vertical draft-tubes, conical plates have been designed to be placed under the tubes and to assist in directing the water horizontally. The present form of hydraucone shows the increased development from the experience of the past. It is interesting to the speaker to contemplate the past fifty-five years, during which so much progress has been made in the efficiency of water-wheels and the designs of draft-tubes.

OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIFORNIA

Discussion*

By Messrs. Charles Evan Fowler and M. M. O'Shaughnessy.†

Charles Evan Fowler,‡ M. Am. Soc. C. E. (by letter).§—The author is to be congratulated on having designed a sea-wall of great excellence, pleasing appearance, and owing to its location at North Beach, of great utility as well. The use of the wave-retarding steps as bleachers for the convenience of the public is unique. The writer has frequently called attention to the desirability of sea-walls designed so as to break the force of waves, and has believed that a face similar to the retarding channels used on the outlets of the Dayton Conservancy dams for reducing velocity, would be very efficacious. The same result has been achieved by the use of the steps or bleachers, and it is to be regretted that it was not thought of for the Galveston sea-wall. This wall, as originally designed and built, did not prove to be satisfactory, but if it had been stepped. as has been done at San Francisco, or had had projections like those of the Dayton channels, the great force of the Gulf hurricane waves would doubtless have been effectually broken. The great velocity of the Gulf hurricanes of 80 to 120 miles per hour, produces waves of much greater height than is usually expected, due to the great surface velocity induced, in connection with a large increase in the water level due to the storm tide created. The force may reach 3 000 lb. per sq. ft. on the face of the sea-wall. This is broken to some extent by a curved face, and the disastrous effect of the drop of the dashing waves on the top of the wall and on the roadway and fill back of it, is often much reduced by a projecting coping.

The wind does not reach as high a velocity at San Francisco, but the ocean waves from the Pacific probably have an intensity of 30 to 60% greater than those on the Gulf coasts. The waves of oscillation on striking the North Beach slope, are changed to waves of translation, so that they strike a direct blow of high intensity. The return of the water is more or less in the form of an undertow of great scouring force, and much of the material in front of the wall will undoubtedly be washed out during storms. It is not likely that these wash-outs will endanger the wall owing to the use of sheet-piling and bulb piles, but they may necessitate the use of considerable rip-rap from time to time. This of course will be objectionable to the people using the beach, but it may be smoothed with sand filling or grouted to some extent without impairing its efficiency. Some groins may also have to be built to save loss of beach area from cross-scouring. The action of the wall during the running

* Discussion of the paper by M. M. O'Shaughnessy, M. Am. Soc. C. E., continued from April, 1924, *Proceedings*.

† Author's closure, except reply to discussion by Charles Evan Fowler, M. Am. Soc. C. E.

‡ Cons. Civ. Engr., New York, N. Y.

§ Received by the Secretary, April 4, 1924.

of the very heavy seas, such as the writer has seen there, will be of great interest to those having to do with sea-works.

The first time the efficacy of a rough sea-wall face was brought forcibly to the writer's attention, was during the filling of the Seattle Tide Flats. This fill was protected on the Puget Sound face by a fir brush bulkhead. The butts of the brush were placed outward on a slope of 45° and to a vertical height of 20 to 25 ft. During severe storms the waves dashing against this face with its almost innumerable projecting points were effectively broken, the water dropping back dead, without harm to the bulkhead and with its scouring force gone.

With future extensions of the Esplanade sea-wall it may be desirable to copy this idea into masonry to some extent, probably along the lines of the Dayton retarding channels. Such a plan might even be combined with the bleachers to make them of greater utility and of an equally pleasing design. If as much thought were given to other engineering works as was given to the Esplanade sea-wall, fewer failures would be recorded.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).†—The writer feels very grateful for the extended discussion of this interesting subject by three members of the Society who have had actual experience in works of a similar nature.

In reply to Mr. Perry, Fig. 2‡ describes the character of the beach previous to the construction of the Esplanade; it is composed of fine sand to a depth of more than 200 ft. to bed-rock. As indicated in that photograph, the sea made successive erosions, eating back as far as 70 ft. behind the wall and causing an unsightly and scragged condition on the beach.

The prevailing winds are from the west and northwest, and the ocean range is more than 4 000 miles across to the shores of Japan. If erosions of the beach had continued after the completion of the wall, it was intended to supplement the structure by building groins about 200 ft. apart. Happily, however, as illustrated by Fig. 7,§ the sand has actually made accretions and, in some places, has covered the steps to within 1 or 2 ft. of the sidewalk level. Presumably, this accretion is largely due to the stepping and the air-pockets caused by the projecting walls rising above the steps, which act as a cushion to the impinging waves.

Capt. Leeds' remark‖ that in this type of work, "the ripest experience and broadest judgment are needed, for every problem is different in its basic conditions—the forces to be reckoned with and the results desired."

Before this design was adopted, a research was made of all protective work in the British Isles and in Europe. A Blue Book of the British Parliamentary Board on Erosions on the Eastern and Southern Coasts of England was carefully perused, and it was noted that solid blocks of concrete, 25 ft. thick and 25 ft. high, were destroyed by wave action, due to undercutting of the founda-

* City Engr., San Francisco, Calif.

† Received by the Secretary, March 13, 1924.

‡ *Proceedings*, Am. Soc. C. E., November, 1923, p. 1849.

§ *Loc. cit.*, p. 1855.

‖ *Loc. cit.*, April, 1924, p. 526.

tion; hence, the elaborate foundation of interlocking concrete piles designed for this structure.

It would be only fair to bear testimony to the assistance received in the design from E. F. Kriegsman, Assoc. M. Am. Soc. C. E., then Assistant Engineer, now Consulting Engineer in San Francisco, and from Mr. W. H. Ohmen, both of whom were enthusiastic and painstaking in their work.

The only previous effort to build a sea-wall in this location was made by the Park Commission, when its Superintendent built a bulkhead with 12 by 12-in. vertical concrete piles immediately south of the Esplanade, in order to save the Chalet Building, a beach resort. This wall will be removed and absorbed in a southern extension of the Esplanade, as it is not architecturally attractive or useful.

Two Chairmen of the Finance Committee of the Board of Supervisors of San Francisco were eager that wooden piling should be constructed as at Longport, N. J., the results of which are plainly shown by Mr. Haupt in Fig. 15.* Quite a discussion was had with the Park Commission as to the position of the wall with regard to tide levels. The Commission was eager to advance it still farther toward the ocean, but the writer insisted on the present alignment, which has been successful. The sea-wall at Coronado, which is made of rip-rap, has cost practically $100 per lin. ft. It is not favorable for pedestrian or general use, as it renders untenable a large area of the most attractive part of the beach.

The writer is thoroughly in accord with the desirability of having many irregular surfaces to split and turn the ocean waves in various directions "while the formation of areas and air-pockets dissipate their forces."

Mr. Haupt has made a most generous contribution to the discussion, and also has described pictorially sea-wall protection in Europe as well as in America. He confirms the failure in Great Britain of concrete walls 20 and 30 ft. deep, due to undercutting and shore erosion. The sea-wall at Longport, N. J., was not similar to the one in San Francisco, as it was founded on wooden piling and was breached the first season after construction.

The general direction of the resultant movements were carefully observed in San Francisco. The small islands outside the Cliff House divide the incoming waves, the southerly parts of which swing directly toward the Esplanade and the northerly sections toward the rocky beach. This explains why such elaborate precautions were made with the footings of this wall, so there should be no undercutting nor failure.

From a structural point of view, the conditions are very difficult for contractors who undertake to build small lengths of wall, as it requires as much plant and equipment for 500 ft. as for 1 000 ft. Money being available only in small units, recommendation for any more construction has been deferred until a substantial sum is available, at least $250 000, when probably 2 000 ft. additional wall could be built in one season.

As built, the structure is standing remarkably well. There are no cracks or fractures of any kind. It is largely patronized by autoists and people who seem to enjoy looking at the ocean from the roadway above it.

---

* *Proceedings*, Am. Soc. C. E., March, 1924, p. 357.

The paper has demonstrated that, on a sandy beach without a rock foundation and by taking the precautions indicated, a sea-wall can be built that will survive the shock of weather and waves and be of lasting benefit to the community.

pers.

nda-
will
the

BEACH EROSION: ITS CAUSES AND CURE

Discussion*

By Henry Clay Ripley, M. Am. Soc. C. E.†

Henry Clay Ripley,‡ M. Am. Soc. C. E. (by letter).§—The writer feels much indebted to the members of the Society who have discussed his paper and gratified at the favorable opinions which are generally expressed concerning its conclusions. In order to emphasize some of these opinions, it may be desirable to repeat them here.

Col. Riché|| thinks that a trial of the writer's plan would be interesting and promises good results.

Col. Dent¶ is glad to know of a system of beach protection that will positively and definitely stop erosion whenever the protection afforded is worth the cost of obtaining it.

Mr. Sherman** concurs in the writer's opinions, mentioning the sea-wall at Seabright, N. J., and the hurdles at Beach Haven, N. J., as evidence. This latter structure is located entirely above the low-water line and is not on a line parallel with the shore; in these respects, it differs radically from the writer's plan.††

Mr. Staniford‡‡ remarks "that the principle of a lateral breakwater beyond low water is not tenable for the Atlantic Coast without undue expense in construction," and "that the offshore bar provided by Nature operates naturally in performing some of the functions of such a breakwater." Thus, although he prefers some other method of beach protection, yet he recognizes that the principle of the offshore breakwater is correct.

In his adverse opinion, Mr. Gelineau§§ gives an interesting description of an experiment which he made on the New Jersey Coast to control the littoral current while permitting wave action to have free play. This is a method directly opposed to that suggested by the writer, and furnishes an instructive lesson to those attempting to control the littoral current by jetties or groins, showing some of the difficulties and uncertainties attending such methods; and, yet, Mr. Gelineau considers that the writer's plan of an offshore breakwater cannot be made successful without some connection with the shore. The absence of any shore connection is an essential feature of the writer's plan

* Discussion of the paper by Henry Clay Ripley, M. Am. Soc. C. E., continued from April, 1924, *Proceedings*.

† Author's closure.

‡ Cons. Engr., Detroit, Mich.

§ Received by the Secretary, March 29, 1924.

|| *Proceedings*, Am. Soc. C. E., April, 1924, p. 539.

¶ *Loc. cit.*, p. 536.

** *Loc. cit.*, p. 541.

†† *Loc. cit.*, March, 1924, p. 371.

‡‡ *Loc. cit.*, April, 1924, p. 532.

§§ *Loc. cit.*, p. 535.

as it allows free movement to the littoral current so as to furnish the material for accretion in back of the breakwater. This littoral movement of material will be generally the only source of supply for the purpose of accretion; but Col. Riché has shown that a large quantity of sand was carried over the sea-wall at Galveston during the storm of 1915 and deposited back of it. Thus, it would appear that during storms an additional amount of accretion back of the breakwater will result from this cause.

Two objections to the use of the offshore breakwater are made by Mr. Gelineau: First, its excessive cost; and, second, the injury it will cause to the bathing beach. As to the cost, he estimates for a structure 15 ft. wide on top, 15 ft. high, and with side slopes of 1 to 1½, approximately $250 per lin. ft., or nearly as much as for the Galveston north jetty with a top width of 24 ft., a height of 28 ft. and with side slopes such as the stability of the wall required. The stone for this work was hauled about 300 miles by rail and was in units as large as 15 tons or more. The actual cost was $255.33 per lin. ft.* As regards the second objection, if such construction, uncomplicated by shore connection or groins, has been injurious to the bathing beach, it is unfortunate that no specific case is mentioned.

Mr. Sherman asks whether the projecting section of the beach back of the offshore breakwater would remain intact in the face of an attack by a 6-knot current, such as prevails on the New Jersey Coast during storms. The answer is emphatically—yes. The movement of material along the beach is the result of the combined influence of both current and wave action. No matter what the velocity may be, as the current is the same in the protected area as beyond it and the wave action is less, more material will be brought into it than will be taken out.

He expresses the opinion that the cost of such a work as the writer has suggested, would be great, and that probably less money, if distributed in a series of groins properly designed and suitably located over the same area, would accomplish the needed protection and extend the beach. If this be true, there is nothing further to be said. It is believed, however, that the cost of an offshore breakwater is greatly exaggerated by those opposing its use. A low structure such as generally will suffice on a gently sloping foreshore, if constructed as contemplated by the writer,† can be made a permanent structure at a cost comparing favorably with that of other methods of protection. Where stone is not available at a reasonable cost, reinforced concrete sheet-piling would be quite satisfactory; or where the teredo is absent, wooden sheet-piling will answer.

When, however, the question is raised as to the certainty of the success of any proposed protection, the offshore breakwater is superior to any other method thus far suggested.

Mr. Collins‡ gives an interesting description of a work which he executed in the Philippines for the protection of the beach at Aparri on the northern

* Report of Chf. of Engrs., U. S. Army, 1897, p. 1800.

† "How to Build a Stone Jetty on a Sand Bottom in the Open Sea", *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1040.

‡ *Proceedings*, Am. Soc. C. E., April, 1924, p. 538.

coast of Luzon. This work seems to be in accordance with the most approved method of applying the groin system. The real test of its success, however, will come later. This coast is subject to an annual season of severe typhoons, and it will be only when the work is subjected to one of these disturbances that its real merit will be disclosed.

Under normal conditions, the beach has a much steeper slope from low-water line toward the land than in the direction of the sea. During a storm, however, when the water rises much above its normal height, this steeper gradient is reduced and the material carried seaward, showing a distinct erosion of the beach. After the storm subsides and normal conditions again prevail, this steeper gradient is gradually restored, giving a distinct accretion to the beach. A series of groins built at this time will give the appearance of great success in restoring the lost beach when, in reality, it had nothing to do with it; sometimes the groins are completely buried out of sight and the accretion extends far beyond, showing conclusively that the groins were in no way responsible for the results.

THE DISTRIBUTION OF INTENSE RAINFALL AND SOME OTHER FACTORS IN THE DESIGN OF STORM-WATER DRAINS

Discussion*

By Messrs. Robert E. Horton and Frank A. Marston.†

Robert E. Horton,‡ M. Am. Soc. C. E. (by letter).§—A study was recently made of rainfall depth-area relations in great storms, based on the data collated by the engineers of the Miami Conservancy District.‖ This discussion will be confined to their figures relating to five great Northern and five great Southern one-day storms. Data giving the maximum rainfall depths over different areas in these storms are contained in Columns (7) to (14), inclusive, of Table 13, which figures differ somewhat from those given for the same storms in the report of the Miami Conservancy District, the difference arising from the fact that a storm may have two or more rainfall centers. In the Miami studies, the mean depths were determined from planimeter measurements of the areas surrounding the principal center only, although there might be a subsidiary center having a rainfall greater than that indicated by the principal contours. In the writer's analysis of these data, the entire area in which the rain exceeded a given quantity was included in obtaining the average rainfall depth for that area, whether the rain occurred around a single storm center or around two or more separate centers. The latter appears to be the more rational method of analysis, with reference to a study of the meteorological relations of rainfall and area in great storms.

Plotting the data given in Columns (7) to (14), inclusive, of Table 13, it was found that the resulting curves of area-mean rainfall depth could be accurately represented in every instance by a general equation:

$$P_{av} = (P_0 + c) \cdot e^{-kA^n} \quad \text{.........................(1)}$$

in which, P_{av} is the average rainfall for one day over an area, A, in square miles; P_0 is the highest measured precipitation at or near the eye or focus of rainfall. Since it sometimes happened that there was no rain-gauge which recorded the maximum precipitation, it often becomes necessary to add a small constant, c, to P_0, in order to obtain the precipitation at the eye or focus of the storm. The quantities, k and n, are constants for a given storm. The values of these constants, as calculated for the ten storms referred to, are given in Columns (4), (5), and (6), of Table 13. As a result of various studies, of which the preceding is an example, it appears that for storms of one day

* Discussion of the paper by Frank A. Marston, M. Am. Soc. C. E., continued from April, 1924, *Proceedings*.

† Author's closure.

‡ Cons. Hydr. Engr., Albany, N. Y.

§ Received by the Secretary, February 18, 1924.

‖ "Storm Run-Off of Eastern United States", by the Engineering Staff of the District, Technical Reports, Pt. V, Dayton, Ohio, 1917.

duration, or more, all the average depth-area relations for individual storms are capable of being represented by curves belonging to the same family, the general equation being Equation (1). This applies only to well developed and relatively intense storms. It is, however, precisely this class of storms which are of the most importance in relation to flood hydrology.

TABLE 13.—AREA DEPTH FORMULAS—ONE-DAY STORMS.

$$P = (P_0 + c)\,e^{-k A^n}$$

(Data from Miami Conservancy District.)

Storm number, Miami report.	Date.	Storm center.	CONSTANTS IN FORMULA.				AVERAGE DEPTH (BY FORMULA) Over areas, in square miles, of:						
			k.	n.	c.	P_0.	1 000	5 000	10 000	20 000	30 000	50 000	80 000
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
		NORTHERN STATES.											
Limiting curve..		....	0.0883	0.24	1.4	14.6	10.0	8.0	7.1	6.2	5.6	4.9	4.2
83	6- 9-1905	Iowa............	0.0196	0.41	1.9	12.1	10.0	7.4	5.9	4.4	3.7	...	...
76	10- 8-1903	New Jersey......	0.0306	0.33	...	11.5	8.5	6.9	6.0	5.1	4.5	...	...
c	5-21-1889	Pennsylvania....	0.00125	0.56	...	8.4	7.9	7.2	6.8	6.1	5.6	4.9	4.1
72	8-27-1903	Iowa............	0.00448	0.55	1.0	11.2	10.0	7.5	6.0	4.3	...	...	...
114	10- 4-1910	Illinois	0.0115	0.39	0.5	8.5	7.6	6.6	5.9	5.2	4.8	4.2	2.9*
		SOUTHERN STATES.											
Limiting curve..		....	0.11200	0.23	...	22.0	12.2	9.4	8.0	6.8	6.1	5.1	4.4
135	10- 2-1913	Texas	0.01150	0.45	2.0	13.0	11.6	8.8	7.2	5.6	4.6	...	...
156	7- 7-1916	Alabama	0.00638	0.45	0.5	11.2	10.1	8.7	7.8	6.7	6.0	5.1	4.2
49	4-16-1900	Mississippi	0.00813	0.45	...	12.7	10.6	8.8	7.7	6.4	5.4	4.5	...
13†	9-26-1894	Florida..........	0.00348	0.54	...	12.5	10.8	8.8	7.5	5.9	4.9	...	...
157	7-15-1916	South Carolina..	0.02940	0.41	...	19.8	11.8	7.4	5.4	3.6	...	...	...

* Observed precipitation. Formula gives 3.5 in.

† Occurred along the coast where the true total area could not be measured and was probably much greater than given. (See page 146 of the Miami report.)

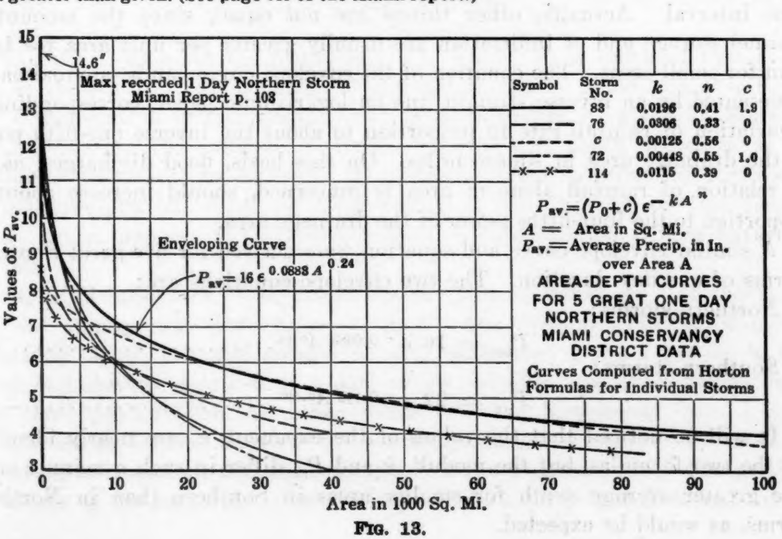


FIG. 13.

The depth-area relation curves for five great one-day Northern storms are shown on Fig. 13. In general, storms having the highest focal intensity, P_0, decrease rapidly in intensity with distance from the focus. General storms with lower focal intensity will frequently give higher average precipitation over large areas. If the individual area-depth relation curves were known for a sufficient number of curves, it would be possible to draw an envelope of the resulting family of curves. Obviously, no single storm will give the true maximum average rainfall to be expected over areas of all sizes. An intense local storm will give maximum average depth over a small area; a large or more general storm giving a less intensity for a small area, will generally be that which will give the greatest intensity over a large area. It would be of great service to engineers if a law or equation could be derived, which would represent even empirically the maximum relative amounts of rainfall to be expected in storms of one day, or other durations, over areas of different sizes. The envelope of the five great Northern storms shown on Fig. 13 represents a first approximation toward this end. This envelope has an equation of the same general type as that of the individual curve, but with different constants. Determination of more accurate curves of the type of this envelope is greatly to be desired.

Engineers have long recognized that maximum discharges from drainage basins having similar characteristics but different areas do not increase in direct proportion to the drainage area, but only as the two-thirds to seven-eighths power. Such area factors are incorporated in most flood-discharge formulas—the McMath and Bürkli-Ziegler formulas, Fuller's flood formula, and others. Apparently, the envelope curve of Fig. 13 represents in the main the law of relation as regards rainfall and size of area and the maximum flood discharge rate to be expected therefrom, since, other things being equal, the flood discharge is nearly proportional to the average rainfall over the area for a given time interval. Actually, other things are not equal, since the amount of channel storage and of infiltration are usually greater per unit area for large than for small areas. The equation of the envelope curve can be approximately represented by an inverse straight line on logarithmic paper, corresponding to a variation of rainfall rate in proportion to about the inverse one-fifth power of the drainage area in square miles. On this basis, flood discharges, as far as relation of rainfall alone to area is concerned, should increase about in proportion to the four-fifths power of the drainage area.

A similar envelope curve and equation were derived for five great Southern storms of one day duration. The two envelope equations are:

Northern storms:

$$P_{av} = 16\ e^{-0.0883\ A^{0.24}} \qquad \text{(2)}$$

Southern storms:

$$P_{av} = 22\ e^{-0.112\ A^{0.28}} \qquad \text{(3)}$$

It will be noticed that the values of the exponent, n, are nearly identical for the two formulas, but the moduli, k and P_0, differ in such a manner as to give greater average depth for smaller areas in Southern than in Northern storms, as would be expected.

This study was made for the purpose of discovering, if possible, the general facts and relations existing in connection with the occurrence of great storms, so as to afford a basis of comparison of statistical data with the known meteorological conditions by which great storms are produced. The formulas given relate to the integral average rainfall over a given area, this being the form in which the data are generally required by an engineer in practical calculation. For the purpose of study of meteorological conditions, it is better to use a formula representing actual rainfall distribution about the eye or focus of the storm, or, in other words, a rainfall depth-radius formula or curve.

The variation of depth of precipitation with distance from center of storm can be expressed as follows, for storms in which the average depth and area are given by the formula:

$$P_{av} = P_0 \, e^{-k A^n}$$

Total volume = average depth × area = $P_{av} \, A = P_0 \, A \, e^{-k A^n}$

Rate of precipitation, $P = \dfrac{d\,V}{d\,A}$, in inches per unit of time, at radius, r, where $A = \pi \, r^2$.

$$d\,V = P_0 \, e^{-k A^n} \, d\,A - (A \, P_0 \, e^{-k A^n} \, . \, k \, n \, A^{n-1}) \, d\,A$$
$$= P_0 \, e^{-k A^n} \, (1 - k \, n \, A^n) \, d\,A$$

or,

$$P = P_{av} \, (1 - k \, n \, A^n) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$
$$= P_0 \, e^{-k A^n} \, (1 - k \, n \, A^n) \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Let $m = 2\,n$ and $a = k \, \pi^n$, then,

$$P = P_0 \, e^{-a\, r^m} \left(1 - \frac{m\,a}{2} \, r^m\right) \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The precipitation becomes zero at a radius:

$$r_0 = \frac{1}{\sqrt{\pi}} \sqrt[2n]{\frac{1}{k\,n}} = 0.565 \sqrt[2n]{\frac{1}{k\,n}} \ldots\ldots\ldots\ldots (7)$$

Equation (6) gives the precipitation at radius, r, in terms of that at the eye of the storm. This equation is rational in that it reduces to $P = P_0$ for $r = 0$ and gives a precipitation, $P = 0$ at some radius, r_0, given by Equation (7). Equation (6) indicates that the precipitation depth decreases as the radius or distance from the eye of the storm increases and that the decrease in actual precipitation depth is proportional to the product of two factors. This, again, appears to be rational from a consideration of the nature of storms.

Rainfall condenses from ascending currents of moist air. If the radius of the atmospheric disturbance is small, the vertical gradient would be relatively steep, but the volume of air transported from a low to a high level and thereby deprived of a part of its moisture will be relatively small, the total precipitation, in other words, will be distributed over a small area. Moist air is deprived by condensation of nearly all its available moisture as it ascends through a height of a few thousand feet. The gradient of ascent of the moist air is proportional to the total height of ascent divided

by the storm radius. For a storm covering a large area, the barometric gradient is more gentle than for a local intense storm, and the gradient of ascent of the moist air is less steep, but the total volume of air involved is much greater. As a result, large storms give greater total precipitation than small ones, but the precipitation rate is often larger on small areas in local than in general storms. A storm is a moving atmospheric system having a motion of translation as well as an internal motion of the air system. The quantity of rain precipitated by a storm in passing over a given area depends on two factors: (1) The rate at which rain is formed within the storm; and (2) the rate of storm travel. The relation of meteorology of storms to rainfall depth is too complex for present consideration beyond the statement of the conclusion, which appears to be justified both by study of weather maps and by rainfall statistics, namely, that, in general, the greater the distance from the eye of the storm the less the horizontal wind velocity and, consequently, the smaller the rate of ascent of the moisture-laden air, and also the less the rate of precipitation. Furthermore, the greater the radius or distance from the eye or focus of rainfall, the less, as a rule, will be the slope or gradient of the ascending air. The quantity of precipitation varies as the product of these two factors, both of which decrease with increased radius or distance from the eye of the storm.

In view of the preceding considerations, the fact that the empirical Equation (6) for relation of radius to rainfall depth contains the product of two factors, both decreasing with the radius, is of considerable interest.

The studies made thus far were based on general storms of one day duration. The question naturally arose as to whether the relations discovered would hold for intense local storms and short time intervals. The appearance of the author's paper was a timely and agreeable surprise, as it furnished much needed data.

Depth-area curves were plotted for all the storms and time intervals included in Table 4* of Mr. Marston's paper, and the constants in Equation (1) were determined. The resulting constants are given in Table 14. Plotting the curves corresponding to these equations, it was found that they were in almost perfect agreement with the observational data in every instance, in other words, depth-area relations for these local storms also follow a general law or can be represented by a family of curves having the same general equation which was found to apply in the case of great one-day storms. In Fig. 14 is shown by plotted points the original data as given by Mr. Marston for two of the storms, whereas the curves shown by solid lines were calculated from the corresponding equations.

No attempt has been made to develop envelope curves for short-interval storms owing to the fact that the data available are insufficient. It should be noted that the formulas based on Miami data are expressed in terms of areas in square miles, whereas those based on Mr. Marston's data are in terms of area in acres. Reduction from one system of units to the other would have the effect of changing the values of the constant, k. The method of determining the constants in these formulas is as follows: Given a set of values of

P_{av} and A for a certain storm, take the value of $P_0 + c$ either as the actual highest measured rainfall at the storm focus, or if the form of the curve indicates that this is too small, extend the curve to its intersection with the axis of Y and use the intersection point as the value of this constant. Call this, for convenience, P_0. Select two pairs of values, P_{av_1} A_1 and P_{av_2} A_2. Then,

$$\frac{P_{av_1}}{P_0} = e^{-k\,A^n_1}$$

$$\frac{P_{av_2}}{P_0} = e^{-k\,A^n_2}$$

From a table of exponentials take out the numerical values of $k\,A^n_1$ and $k\,A^n_2$ corresponding to the known values, $\frac{P_{av_1}}{P_0}$ and $\frac{P_{av_2}}{P_0}$. Call these numerical values Z_1 and Z_2. Then,

$$k\,A^n_1 = Z_1 \text{ and } k\,A^n_2 = Z_2$$

Dividing,

$$\left(\frac{A_1}{A_2}\right)^n = \frac{Z_1}{Z_2}$$

from which n is easily determined by logarithms. The value of k is next found from the formulas for Z_1 and Z_2.

TABLE 14.—INTENSITY AREA FORMULAS.

Constants in the Depth-Area Relation Formula:

$P_{av} = (P_0 + c)\, e^{-k\,A^n}$.

P_{av} = Average Intensity over Area A.

A = Area, in Acres.

(Data from Boston Metropolitan District, by Frank A. Marston, M. Am. Soc. C. E.)

Storm.	Duration, in minutes.	DERIVED CONSTANTS.			P_0.
		c.	k.	n.	
(1)	(2)	(3)	(4)	(5)	(6)
8	15	0	0.00304	0.56	3.20
6	"	....	....	....	....
7	"	0	0.00226	0.53	2.48
18	"	0	0.00449	0.57	2.16
20	"	0	0.00131	0.61	1.56
2	"	0	0.00244	0.58	1.56
1	"	....	....	....	....
4	"	0	0.00105	0.59	1.12
16	"	....	....	....	....
15	"	....	....	....	....
9	30	....	....	....	....
10	"	0	0.00256	0.55	2.88
19	"	0	0.00375	0.57	1.70
3	"	0	0.00027	1.015	0.86
5	"	....	....	....	....
17	"	....	....	....	....
11	45	0.03	0.00258	0.51	2.17
12	"	0	0.00154	0.57	1.72
13	60	0	0.001185	0.56	1.72
14	"	0.02	0.00148	0.55	1.29

* *Proceedings*, Am. Soc. C. E., January, 1924, p. 29.

The main object of these studies has been to develop the meteorologic aspects of the subject with a view to laying a foundation whereby results as nearly rational as possible might be derived from statistical studies. A practical application of the formulas, however, lies in the fact that it is possible by their use to determine the average rainfall depth over any area for a storm of any assumed characteristics. Incidentally, it often happens that a storm having the required characteristics to produce a maximum rainfall on a given area may never have been recorded; still, the magnitude of its rainfall can be closely approximated from data for other storms of record. For example, if two storm records show, for the first, $P_0 = 12.0$ in. in 24 hours and $P_{av} = 9.0$ in. on 2 000 sq. miles, and for the second, $P_0 = 8.5$ in. at the focus of the storm and 6.1 in. on an area of 20 000 sq. miles, then the probable maximum rainfall to be expected on an area of 10 000 sq. miles may be determined by using the data given to find the constants in the rainfall depth-area equation. The resulting equation in this case will be, $P_{av} = 16\ e^{-0.0883\ A^{0.24}}$, from which it is found that a rainfall of 7.2 in. may be expected on an area of 10 000 sq. miles. This is based on actual data for two of the greatest storms analyzed by the Miami Conservancy District. It is evident that there is a storm of a certain particular radius, which will give a greater precipitation over a given area than either a storm of smaller radius and higher focal intensity, or a storm of larger radius and lower focal intensity.

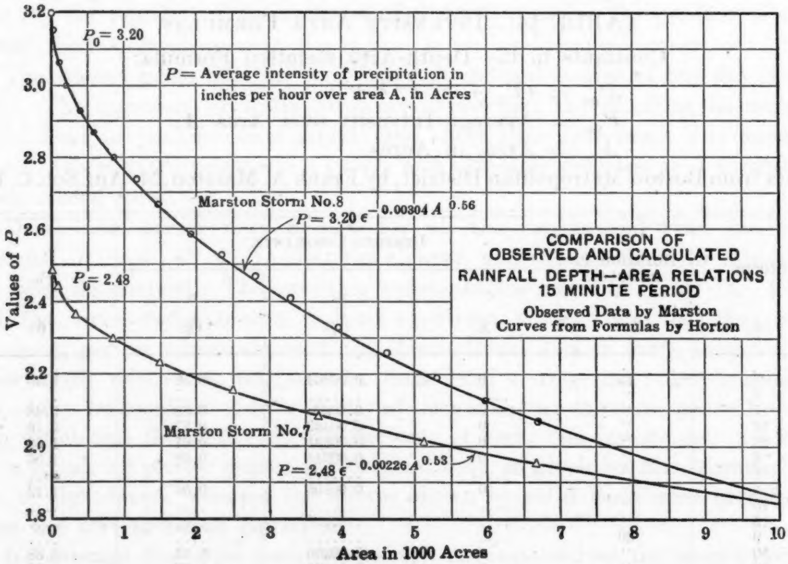


FIG. 14.

It is to be hoped that further contributions to the study of rainfall depth-area relations may be made. When a sufficient body of data is available, the envelope curves representing the maximum average rainfall depth to be expected over an area of any given size can be developed and an important advance in the determination of maximum rainfall allowance will have been

attained, not only for storm sewer design, but in connection with other hydraulic works.

FRANK A. MARSTON,* M. AM. SOC. C. E. (by letter).†—Through the courtesy of Allen Hazen, M. Am. Soc. C. E., the author was privileged to examine two papers‡ by A. Frühling of Dresden, Germany. Inasmuch as the material contained in these papers has not been published elsewhere, it may be of interest to refer briefly to the studies on which Frühling based his discussion of the distribution of intense rainfall, referred to in the writer's paper.§

From 1886 to 1892, observations were made at three stations in Breslau, separated by distances of 4 250, 2 750, and 4 500 m., respectively. As a rule, the progress of a storm was such that showers reached and passed one station before arriving at the second or third stations, and generally only touched one station. Cases when rain fell simultaneously at two or three stations were the exceptions.

With these observations as a basis, Frühling assumed that, for lack of a better basis, the decrease in intensity of rainfall, with the distance from the center of the shower, could be represented by a parabola in which h, the ordinate, was the mean greatest intensity at the point of observation. At a point where L, the abscissa, was 3 000 m., the ordinate was $\frac{h}{2}$.

From the few observations given, it is apparent that Frühling made an extensive study of the subject along lines considerably in advance of his time. Because his formula is too general and fails to take into account the frequency of a shower, it cannot be used practically. It is interesting historically to note that Frühling refers to similar studies made by the late Emil Kuichling, M. Am. Soc. C. E., at Rochester, N. Y., in connection with the design of a sewerage system.

The program of experimental work on run-off, described by Mr. Allen,‖ is of much interest to all engineers who have to design storm-water drains. Such investigations, carried on under expert supervision, should be encouraged and supported in every possible way. In the larger cities like New York, Chicago, St. Louis, and New Orleans, there are municipal organizations that could be utilized to collect much necessary data. Unfortunately, the expenditure of funds is often controlled by officials who do not appreciate the value to be derived from research work of this character.

It is hoped that Mr. Horner¶ will publish the results of the research work in St. Louis, where he has devoted much study to scientific methods for the design of storm-water drains.

Mr. Hazen's suggestion** that studies be made of the rate of precipitation on a given area, as a means of determining "the falling off in rate of

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

† Received by the Secretary, March 28, 1924.

‡ "Ueber Regen- und Abflussmengen für städtische Entwässerungskanäle", *Civilingenieur*, XL. Band, 6-7 Heft (1894).

§ *Proceedings*, Am. Soc. C. E., January, 1924, p. 25.

‖ *Proceedings*, Am. Soc. C. E., April, 1924, p. 543.

¶ *Loc. cit.*, p. 557.

** *Loc. cit.*, p. 558.

any specified frequency", has appealed to the writer as worthy of trial, and already material progress has been made in compiling the 5-year records of the Boston District, for a specific area. If such a method proves satisfactory, it will make possible the use of records for which closed isohyetals could not be drawn, and, in time, will also permit direct determination of frequency. The area of closed isohyetals should also be measured, as described in the paper,* and recorded to furnish more complete data showing the reduction in average intensity with increase in area.

In the paper, the district for which closed isohyetals have been drawn and the data recorded, covered an area of about 17 000 acres, which is relatively small. Whether or not the rate of precipitation falls off more rapidly with increasing area than has been indicated, can only be determined by additional studies. It is possible that studies on a specific area of 1 000 to 5 000 acres may indicate that the frequencies referred to are too low.

The writer has called attention to the fact that the 38 years of frequency records of the Chestnut Hill gauge were used in the measure of frequency of the downpours studied, for want of better or more applicable data. Consideration was given to the possible use of Professor Meyer's valuable formulas showing the intensity of rainfall and frequency, for a group of cities, including Boston.

The formula for Group No. 3, in which Boston is located, was based on the records of recording gauges in nineteen cities, among which were Knoxville, Memphis, Cairo, Indianapolis, Cincinnati, St. Paul, etc. These cities are widely separated and are affected by many climatic and topographic conditions not comparable with those of Boston. In no case did the records from any one gauge cover more than 19 years (1896–1914, inclusive). It is possible that 19 years is too short a period from which to determine that a certain city should be placed in Group No. 3, instead of No. 2 or No. 4. Necessarily, some cities will be close to the arbitrary dividing line between groups, and could be placed in one just as well as in another.

Professor Meyer's grouping segregated those cities for which the average intensities of precipitation for a 5-min. period of 1-year frequency were approximately the same (3.84 in. per hour). An examination of the table† for Group No. 3, showing intensities for a 30-min. period and 10-year frequency, which is more applicable to drainage problems, indicates that the results of the groupings might have been quite different if sufficient data had been available to make the segregation on the basis of a longer period of duration and higher frequencies.

In view of the foregoing considerations, it seemed preferable for the needs of the paper, to use the 38-year record from one of the gauges in question (Chestnut Hill gauge) rather than the general formula. It is recognized, however, that for many purposes data based on a number of stations rather than on the record of one station may be safer. The use of the general formula in this case appears to be open to greater criticism than the use of the records of one of the Boston gauges.

---

* *Loc. cit.*, January, 1924, p. 31.

† Table 14, p. 178, Meyer's "Hydrology".

A study has been made of the combined records of the Chestnut Hill gauge (38 years, 1879–1916, inclusive) previously referred to, and those of the U. S. Weather Bureau gauge at the Boston Post Office (26 years, 1891–1916, inclusive), making a total of 64 station-years of record. No continuous long-time records are available for any other of the Boston gauges. The frequencies obtained from the records of the 64 station-years varied but little within the limits used in the paper from those of the Chestnut Hill gauge alone.

If, in time, a long series of observations can be obtained for a particular area in the Boston District, the frequency of occurrence can then be determined with greater accuracy; until then, however, some other method must be used.

The writer has given the data for average intensity of precipitation and area covered, without complications as to frequency, so that it is possible to apply such frequencies as each investigator may deem best suited to a specific problem.

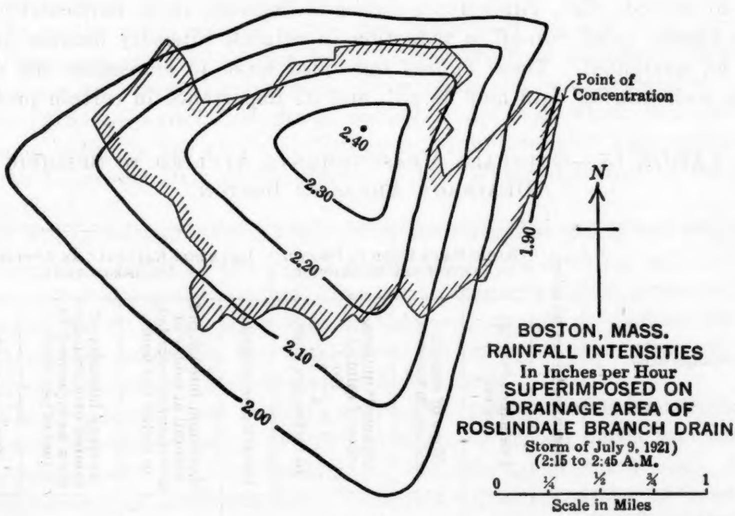


FIG. 15.

Mr. Horton* has contributed an interesting and valuable discussion of the relation between depth or intensity of rainfall and area covered, for storms of long duration, and has added data derived from the Boston observations. The fact that the data when plotted follow so closely to the theoretical curves, indicates that the measurements probably are well within reasonable limits of error. It is also important to note in Table 14 of Mr. Horton's discussion that, in only two cases, Storms 11 and 14, was it necessary to add the constant, *c*. Apparently, the maximum intensity recorded by one of the gauges was, with only two exceptions, the maximum of the downpour. In the case of the two exceptions, the maximum of the downpour was

* See p. 660.

only 0.03 and 0.02 in. per hour, respectively, higher than the maximum recorded. This evidence seems to controvert the conclusions reached by Professor Meyer.* The mathematical studies presented by Mr. Horton should be of value in future researches.

Edgar S. Dorr, M. Am. Soc. C. E., has furnished the writer with outlines of certain Boston drainage areas on which have been superimposed the rainfall intensity diagrams most closely corresponding to the basic assumptions used in the design of the drain. The drainage area of the Roslindale Branch of Stony Brook, shown in Fig. 15, is a typical illustration. The data used in design by the engineers of the Boston Sewer Service, together with similar data computed from the observed rainfall diagram, are given in Table 15. On the basis of this observed rainfall, the average intensity of precipitation over the drainage area is about 7% less than the maximum intensity assumed in the design as uniform over the area. Similar reductions are shown by the data for other downpours and drainage areas given in Table 15.

Although these figures are too meager to afford conclusive deductions, they do indicate that, with certain classes of drainage areas, particularly large areas having quick run-off, a reduction in rainfall intensity because of area may be warranted. These figures may also serve to emphasize the use to which such rainfall data may be put, and its importance in certain problems.

TABLE 15.—RAINFALL MEASUREMENTS APPLIED TO SPECIFIC DRAINAGE AREAS IN BOSTON.

Drainage area.	Storm reference number.	Basic Data Used in Design by Boston Sewer Service.			Data for Rainfall as Applied to Drainage Area.			
		Drainage area, in acres.	Time of concentration, in minutes.	Assumed* uniform rate of rainfall, $i = \frac{150}{t + 80}$, in inches per hour.	Duration, in minutes.	Maximum intensity of rainfall at center of downpour as recorded, in inches per hour.	Average intensity of rainfall over drainage area, as measured, in inches per hour.	Percentage of reduction in intensity due to area.
Roslindale Branch of Stony Brook........	9	1241	32	2.42	30	2.40	2.18	7.2
Roslindale Branch of Stony Brook........	10	1241	32	2.42	30	2.38	2.08	11.2
Dorchester Brook.......	9	891	35	2.31	30	2.40	2.21	6.0
"	10	891	35	2.31	30	2.38	2.16	7.7
Spring Street Brook....	11	606	47	1.95	45	2.17	2.03	4.9
"	12	606	47	1.95	45	1.72	1.59	7.0

* By Edgar S. Dorr, M. Am. Soc. C. E., 1892.

The writer is grateful for the interest shown in the subject and for the valuable discussions contributed. If this work may serve as a stepping stone to the ultimate collection of adequate data on the distribution of intense rainfall, not only in one section of the country, but in several, the labor involved will have been well repaid.

* *Proceedings*, Am. Soc. C. E., April, 1924, p. 551.

THE HYDRAULIC DESIGN OF THE SHAFT SPILLWAY FOR THE DAVIS BRIDGE DAM, AND HYDRAULIC TESTS ON WORKING MODELS

Discussion*

By Messrs. Allen Hazen, Karl R. Kennison, H. de B. Parsons, H. F. Dunham, Eugene E. Halmos, and Charles W. Comstock.

Allen Hazen,† M. Am. Soc. C. E. (by letter).‡—This paper is an interesting account of a novel construction. The writer had the pleasure of inspecting the work that is described while it was under construction, as well as the model of the spillway in operation. He is in no doubt as to the merits of this type of spillway, and commends the excellent experimental work.

The mathematics used to obtain the water down an easy slope are rather appalling; a simpler calculation might answer practical purposes. The mathematics are good mental gymnastics—interesting and possibly helpful.

In 1918, the writer designed a temporary spillway which had enough in common with the one at Davis Bridge to be of interest. This spillway was for the Calaveras Dam of the Spring Valley Water Company of San Francisco, Calif.

It will be remembered that the top of the Calaveras Dam§ had slipped into the reservoir, carrying with it the outlet tower and covering with débris the upper end of the outlet conduit. The most pressing problem presented was to draw off the 70 ft. of water then in the reservoir and to find an outlet for further water quantities, for if this were not promptly done, what was left of the great dam would be overtopped and destroyed.

The first remedy was to drive a new and independent tunnel through the adjacent hillside. While this was under way, a second outlet was sought to give added safety and convenience in the subsequent operations. The old outlet conduit, of ample capacity, was buried by the slip under 70 ft. of débris. Two small channels through massive concrete at its upper end had been left to serve in draining the bottom of the reservoir; their combined area was 41 sq. ft. A caisson well, 16 ft. in diameter, with walls 2 ft. thick, was sunk through the débris exactly on the site of the old outlet shaft to the foundation, which proved to be intact. A short connecting tunnel was then driven through the débris, connecting the bottom of this well with the previously mentioned openings in the head of the old conduit. These openings limited the capacity. As first built, the caisson well was 80 ft. deep. It was supposed that water would flow over its edge and fall into the well and accumulate in it, but always flow-

* Discussion of the paper by Ford Kurtz, M. Am. Soc. C. E., continued from April, 1924, *Proceedings*.

† Cons. Engr. (Hazen & Whipple), New York, N. Y.

‡ Received by the Secretary, March 1, 1924.

§ *Engineering News-Record*, Vol. 80, p. 679; Vol. 81, p. 1158.

ing according to the head through the openings to the old conduit. The conduit itself had capacity to carry more water than could be taken to it. It was expected that between 1 600 and 2 000 cu. ft. of water per sec. could be discharged. Under existing conditions, with many feet of free-board, there was space to hold temporarily the run-off from a flood without overtopping the dam, and this outlet, or the new one, or both together, would carry it off gradually.

In practical operation, the spillway worked admirably, but not exactly as expected. When the water began to go over its edge, it broke into a fine spray, and, falling with a high velocity, carried air downward with such force that the current of air going through the old conduit and coming out hundreds of feet below, was greater than a man could face.

In the five years of operation, the water has never been deep enough to submerge the inlet completely, and the downward flow of water has always been accompanied by a flow of air. When the writer last saw it, after some fairly large floods, the concrete in the narrow passages at the bottom showed no erosion from use. This was in contrast with earlier experience, before the slip of the dam, when, under much lower heads, with solid water, there had been considerable punishment of the very heavy and hard concrete in these same narrow channels.

As a result of this experience, the writer was led to believe that the best design for such a spillway is in the form of a gigantic air-pump. The air, intimately mixed with the water, gives it elasticity and prevents the making and breaking of vacuum with accompanying shock, which is so destructive to outlet structures where solid water flows at high velocities.

G. A. M. Elliott, M. Am. Soc. C. E., Chief Engineer of the Spring Valley Water Company, has since raised the flow line of the reservoir and the shaft of the spillway, so that the drop is now 100 ft. He will, it is hoped, give added particulars of its operation to date.

It may also be of interest to note that the principal spillway of Lake Chabot of the East Bay Water Company is a tunnel. This tunnel spillway, built in 1889, is 1 846 ft. long, horseshoe-shaped, and 10 ft. wide, and 10 ft. high throughout. The slope is steeper at the upper end, but notwithstanding this, the limiting point of its capacity is undoubtedly at its entrance. This entrance originally had square corners of cut stone; these were afterward cut off and replaced with concrete, rounded to ease the entrance and increase its capacity, now estimated at 3 000 cu. ft. per sec., more or less.

It may be noted that shortly after this Calaveras experience, the writer designed a spillway to operate in much the same manner for the San Pablo Dam near Berkeley, Calif., but as this was a new design, some developments and improvements were possible.

A preliminary design was also made in 1918 for a permanent spillway for the Calaveras Dam. This was a study only, and the plan was not adopted. This design was intended to avoid permitting large quantities of water to drop 200 ft. or more on rock so soft as to be eroded by water at high velocities. The proposed outlet tunnel was about 2 700 ft. long with a drop of about 350 ft., the tunnel being 18 ft. in diameter, except that it was enlarged in its upper

end where the full velocity could not be developed. This tunnel was intended to carry a possible flood of 20 000 cu. ft. per sec. The edge of the circular inlet basin was 120 ft. in diameter. If it should ever be built, it would no doubt result in a very considerable change in the landscape near the outlet of the tunnel, but as this is far away from the dam, in wild country, and separated by large hills of hard rock, no danger to the dam would result.

All the experience with tunnel spillways so far has been favorable, and the writer believes that their use will prove advantageous in many situations that render open spillways of the usual construction difficult or expensive.

KARL R. KENNISON,* M. AM. SOC. C. E. (by letter).†—A most significant fact in connection with a shaft spillway for an earth dam, such as that at the Davis Bridge site, is the positive limitation of the capacity, due to the fact that the critical discharge is throttled by the shaft and tunnel and is independent of the spillway crest.

In this case, the rating curves of the model indicate that the limit will be about 34 000 cu. ft. per sec., or 185 cu. ft. per sec. per sq. mile of total drainage area and 220 cu. ft. per sec. per sq. mile of area below the Somerset Reservoir. A discharge at a higher rate than this could occur only by overtopping of the earth dam, which would cause failure. Hence, it would be most interesting from the standpoint of design to have recorded the character of the water-shed and its run-off and the computed effect of the reservoir surface areas in reducing flood peaks.

This limit of the discharge to a rate which cannot be exceeded under any consideration presumably has been conservatively applied in this case, when the probable effect of the reservoir area is taken into account. The writer calls attention to it because of its importance in the general problem of shaft spillways.

In determining this fixed limit of the capacity in designing such a spillway, the question naturally arises as to the cost of increasing it. Can the discharge be increased, say, 10 or 20 or 30% at small expense in order to obtain added safety? In general, there are two ways of doing this, the feasibility of which would have to be determined for the case in hand:

1.—By a considerable enlargement of the short vertical shaft at small added cost so that high velocities can begin in the straight tunnel with a high entrance coefficient.

2.—By a location and design of the discharge end of the tunnel so as to recover as much as possible of the velocity head; this method of increasing the discharge would be particularly effective under a low total head.

The details of the design of the spillway crest itself, since they have no important bearing on the discharge capacity, would naturally be governed largely by practical considerations involved in the operation of stop-logs or gates, or the care of ice, etc. The exact shape of the crest to pass the given discharge at a low head is of little consequence.

* Cons. Engr., Boston, Mass.

† Received by the Secretary, March 4, 1924.

One of the reasons advanced for not building the ordinary type of spillway is that the steep slopes of the canyon walls would make the construction expensive on account of excessive excavation. This is also an argument against building the crest around the shaft in the form of a circle of such large diameter. Where the canyon walls are steep, the spillway crest could be built in straight sections along the contour of the hill running into a smooth bell-mouth shaft entrance. This would result in a cheaper concrete structure for the crest and for the stop-log bridge and handling apparatus, and yet it could easily be made to produce the same rating curve for the discharge of floods, and would involve no great difficulties in hydraulic design.

Practically none of the head saved by the exact circular crest is applied to increasing the discharge.

The author's treatment is an interesting and instructive addition to the literature on the subject.

H. de B. Parsons,* M. Am. Soc. C. E. (by letter).†—The author is to be commended for contributing this paper so replete with detail calculations; likewise, the New England Company for undertaking the interesting model experiments.

The writer has always favored obtaining information from models, and believes that benefit would accrue to both owners and constructors by making greater use of experimental research. Models should be constructed to as large a scale as circumstances will permit, and results obtained by observation and measurement should not be extended beyond the limits of experimentation with the model. Water, air, and barometric pressure are the same in experiment and practice, but friction, eddies, and similar phenomena will not be directly proportional to the model scale adopted. Corrections for model scale will be difficult to apply accurately.

The amount of air drawn in by the spillway model will not be relatively proportional to the amount entrained by the water in the actual spillway, as the velocities and suction effects of water entering the shaft are so different in the two cases. The suction effect of a flow of water into a funnel that is transferred into a shaft will change rapidly with velocity and depth of water. As the depth and velocity increase, the entrained particles of air will become rarefied, that is, a partial vacuum will be produced. This will be changed later to positive pressure (compression), as velocity is lost by friction in the shaft and tunnel. At some stage of flow—probably not indicated in the model—this change of air from vacuum to compression in the shaft may set up a vibratory action. The rock through which the shaft is excavated may be massive enough to suppress all signs of vibration, but it will exist and may affect the lining, should any part become loosened.

Ice cakes or logs, carried into the spillway, will pass into the shaft at high velocity, and are likely to strike heavily against the lining of the bend. Possibly some precaution, not referred to in the paper, is being taken to prevent trouble from this cause. A compound curve for the shaftway bend

* Cons. Engr., New York, N. Y.

† Received by the Secretary, March 11, 1924.

would have been better than the circular curve, but, perhaps, the conditions at the site prevented this refinement, as they did the use of a longer radius for the curve.

It is to be hoped that after the shaft spillway is finished, the author will contribute another paper telling of actual observations under both moderate and flood discharges of water.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).†—A brief visit to the Davis Bridge Dam in the fall of 1923 led to a feeling of surprise because the unfinished up-stream and down-stream slopes were not terraced or rolled and were very steep, the material being dumped from trains. The relation of the ends of the middle or pool area to the mountain sides was not clear. The author's paper is on the spillway, at that time partly established, and two of its features seem to invite mention to the exclusion of other phases of the work.

For a considerable part of its length, the axis of the spillway is vertical. The tunnel-curve section and somewhat more is in the solid rock of the hillside. If the surrounding material other than rock in place were to be taken away, the upper part of the structure would be in the air and unsupported except by the wall of the shaft. In appearance, then, the spillway would closely resemble the author's Fig. 3.‡ What provision was made to support the outer edge or lip of the spillway? Does it rest on a circular wall of masonry or concrete 160 ft. in diameter springing from the rock below and of the variable height necessary to meet the horizontal plane which defines the overflow? If so, what are the dimensions of that wall? How was the space between that outer wall and the shaft wall filled or treated? Gravitation and the quality of the material of the adjacent formations have helped to shape the hillside slopes of that valley; changes in slope or material will not exempt either from the active forces that have prevailed in the past.

In Vermont, not more than sixty miles north of the Davis Bridge Dam, and on ground about 700 ft. lower, there are several large shafts open to the sky and of depths comparable with that of the spillway shaft. Many tons of coal are lowered into those shafts and burned to melt the ice formed in the winter season, to make the frozen rock "lift better", and to enable the workmen to be more comfortable in summer and winter. The temperature of the rock is very low. There are places in that part of the State where ground or well water registers less than 40° Fahr. in the summer season. In winter, the cold air constantly descends and upward currents carry away increments of heat; in summer, this process is not reversed. Little heat from the sun finds its way to the bottom at any season. Those who have been thrilled by a view of the American Fall from Inspiration Point and who, from the same viewpoint a week later, were unable to see any water at that Fall, will realize more fully the effects of continued low temperature in the latitude of Niagara.

Eugene E. Halmos,§ M. Am. Soc. C. E. (by letter).‖—The hydraulic and structural design of shaft spillways should receive serious study by engineers

* Civ. and Hydr. Engr., New York, N. Y.

† Received by the Secretary March 17, 1924.

‡ *Proceedings*, Am. Soc. C. E., December, 1923, p. 1959.

§ Chf. Engr., Parklap Constr. Corporation, New York, N. Y.

‖ Received by the Secretary, March 26, 1924.

interested in hydro-electric developments. The author deserves a great deal of credit for bringing this subject before the profession.

The writer believes that, while this method of disposing of flood waters seldom is economical on wide streams and, especially, in connection with earthen dams, it will be frequently used on narrow streams flowing in deep valleys. In the design of a number of power plants for hydro-electric projects on both Eastern and Western streams, necessitating the construction of high dams built in narrow canyons, he has found, with few exceptions, that the shaft spillway presented the cheapest and most satisfactory means of flood disposal. One or more diversion tunnels built to take care of the river flow during construction assures not only reasonably continuous building operations, but makes it unnecessary to break up the work into small sections. One single coffer-dam can be provided for the whole dam with a corresponding important reduction in cost. With proper foresight in the design, such diversion tunnel or tunnels can be given dimensions to correspond to the requirement of the shaft spillway, thereby incorporating in the permanent structure a piece of work needed for construction purposes.

The writer believes that the method used at Davis Bridge for accelerating the water into the shaft by a "free" fall on the converging apron of the spillway is open to criticism. In his opinion, better results can be obtained by building up the column of water above the shaft opening to a height which will give the necessary accelerating head. This is easily accomplished by the construction of a standard spillway dam and by excavating the enclosed area to a depth and to slopes determined by the static head needed to produce the velocity required in the shaft. By this method, full advantage can be taken of the pressure represented by the weight of the atmosphere, thereby considerably diminishing the necessary depth of the water above the opening and correspondingly reducing the volume of the excavation. The tests described by the author plainly indicate, as it is also remarked in the paper, that the maximum capacity of the spillway was reached when the shaft acted as an orifice. There seems to be no reason, therefore, to design the shaft for a less efficient condition.

The siphon or draft-tube action which is inherent to the upper part of a vertical pipe flowing at capacity, and which, curiously enough, is mentioned in the paper as unexplained and as not having been anticipated, will result at Davis Dam, as demonstrated by the tests, in sucking air into the shaft and causing tremor and disturbed flow at all discharges except those above the designed capacity.

Producing the accelerating head by pressure, as proposed by the writer, will prevent air entering the conduit for a considerable range of discharge volume. It is even possible to obtain this condition for all discharges by dividing the shaft into several conduits and controlling their inlets with gates. The flow then will be smooth and less likely to injure the concrete lining of the shaft and of the tunnel than would the eddying, surging flow resulting from the freely falling jet.

Charles W. Comstock,* M. Am. Soc. C. E.—The speaker's first impression on reading this interesting hydraulic study was that 27 000 sec-ft. seemed a

* Engr., Dwight P. Robinson & Co., Inc., New York, N. Y.

very large spillway capacity to provide for floods from a drainage area of only 180 sq. miles. With this in mind, he made a calculation to determine the necessary time for the water surface to rise from the elevation of the spillway sill to a level about 8 ft. higher.

The spillway without piers or flash-boards was used; the area of the reservoir surface was taken as 2 000 acres (assumed constant throughout the rise); a constant inflow of 27 000 sec-ft. was assumed; and the author's coefficient for the weir was accepted as correct. The calculation showed that about 21.5 hours would be required for the water to reach a depth of 7.84 ft. above the spillway sill. Theoretically, the time required to reach a depth of 8 ft., at which the outflow would equal the inflow, would be infinite. During this time the inflow would be approximately 48 000 acre-ft., of which about one-third would be stored, the remainder passing through the spillway. This volume of water is equivalent to a run-off of 5 in. from an area of 180 sq. miles, and would probably represent a precipitation of 7 or 8 in., certainly an enormous, although not an unprecedented, rainfall.

The amount of money which may properly be expended to provide against remote contingencies is a matter of judgment, in which the engineer is responsible only to his client, and concerning which he should not be criticized because others hold different views. The most that can be done in discussing such a point is to bring out as many expressions of opinion as possible, leaving to each man who may encounter similar problems in the future the privilege of making a decision for himself.

The second point which seems to call for comment is one which is not peculiar to the shaft type of spillway, namely, the use of flash-boards. The spillway is presumably designed with the same care that is given to any other part of the work, and embodies the engineer's best judgment as to margin of safety. To obstruct it with flash-boards for the sake of increased storage capacity is analogous to providing a steam boiler with a safety valve and then tying it down to permit higher pressure, on the assumption that the attendant will watch the gauge and release the valve if the danger point approaches. Safety valves, spillways, and all types of automatic protective devices are intended to provide against the absence, inattention, or disability of attendants, as well as other unforeseen happenings, and the speaker believes that the engineer, having used his best judgment in determining the proper margin of safety, should have the courage of his convictions and not permit encroachment on that margin, however attractive and desirable an increase of storage, speed, or pressure may seem.

There has recently come to the speaker's attention the case of a reservoir located near the head-waters of a stream where a regular practice of using flash-boards in the spillway was in force. Some time ago, following heavy rains, it became necessary to remove the flash-boards. The dam in question suffered no damage, but during this same flood period an earth dam farther down the stream was over-topped and destroyed. The owners of the upper reservoir are now being sued for heavy damages, the charge being that the excess water released by removal of the flash-boards, added to the already

swollen stream, caused the damage below. Whether or not this is true, or whether the lower dam might have been topped by the natural flood without the addition of reservoir water, it is certain that the upper reservoir would not have been open to the charge and under the necessity of defending a costly lawsuit had the spillway been clear and unobstructed at all times.

ANALYSIS OF THE STRESSES IN THE RING OF A CONCRETE SKEW ARCH

Discussion*

BY MESSRS. A. H. BEYER, A. G. HAYDEN, E. H. HARDER, JOHN SANFORD PECK, AND J. CHARLES RATHBUN†

A. H. BEYER,‡ M. AM. SOC. C. E.—The speaker will confine his remarks to the assumptions made in the development of this theory of the skew arch and to the practical application of the formulas derived.

The mathematical analysis of the skew arch as developed by Professor Rathbun is based on the common theory of flexure as applied by engineers to the elastic arch and other statically indeterminate structures. The six fundamental equations (Equations (1*a*) to (1*f*)),§ which determine the six unknown straining actions at the crown of any skew arch, have been developed in a manner which can be readily followed by those familiar with the theory of elasticity.

In the development of the elastic theory of the right arch, the deformation of the arch ring resulting from the shearing forces and torsional moments has been neglected in the two dimensional stress analyses in common use. In the skew arch, under certain conditions, the shearing forces and torsional moments become important, and any theory failing to take into account the deformation of the arch ring from these sources must be considered, at best, an approximate one.

The resultant deformation in a material due to shearing forces can be expressed by a simple mathematical relation only when the resultant shearing stresses are uniformly distributed throughout the mass. This assumption of uniform stress distribution has been made by Professor Rathbun in setting up Equations (27*b*) and (27*c*).‖ The error introduced from this assumption, however, is small. In Equation (27*b*), although the shearing stress distribution is far from uniform, the intensity varying from zero at the extrados and intrados to $\frac{3}{2}$, the average at the neutral surface, the total transverse shear in the direction of the u-axis is usually very small in any arch.

In Equation (27*c*), which gives the deformation due to a shearing force, T_z, acting parallel to the z-axis, the assumption of a uniform shear stress distribution is more nearly true and, consequently, the error introduced by

* Discussion of the paper by J. Charles Rathbun, M. Am. Soc. C. E., continued from April, 1924, *Proceedings*.

† Author's closure.

‡ Associate Prof., Civ. Eng., Columbia Univ., New York, N. Y.

§ *Proceedings*, Am. Soc. C. E., February, 1924, pp. 139-140.

‖ *Loc. cit.*, pp. 159-160.

assuming it to be uniform is not large, although, under certain conditions, the shearing force, T_z, may be very large in a skew arch.

In the development of Equation (27*f*),* Professor Rathbun has found it necessary to equate, for a part of the arch ring, the angular deformation resulting from torsional moments. The mass under consideration, subjected to this torsion, is a parallelopiped the width, b, of which is many times its thickness, t. There is a question whether Equation (2)† can be applied with any high degree of accuracy to a structure such as an arch. The effect of curvature in the arch ring and of the fixed ends at the springing line plays a most important part in torsion, and the effects therefrom have been entirely ignored in Equation (2), which was intended to apply only to a rectangular bar without end restraint. Professor Rathbun was fully justified, however, in making these assumptions, for without them no mathematical analysis is practical; moreover, the errors introduced are small.

The mathematical analysis developed in this paper is primarily of value to the Engineering Profession, in that it proves conclusively that, for vertical loads symmetrical about the arch axes, the stresses in a skew arch can be determined by making a co-planar analysis of a lamina taken parallel to the spandrel walls of the arch. This is of importance in view of a paper,‡ in which Dr. E. Fischer tried to show, but without producing any proof, that for vertical loads the reactions lie in the vertical plane, which is normal to the abutments and passes through the center of gravity of the vertical load. As the theory thus developed by Dr. Fischer appears in part to account for the high stresses that are known to exist at the haunches of certain skew arches near the obtuse corners, it received some approval. The mathematical analysis made by Professor Rathbun shows that Dr. Fischer's assumptions are approximately true only for a simple skew slab, resting on but not clamped to its supports. If the ends of such a slab are rigidly fixed, the same mathematical analysis shows conclusively that the load is transferred to the supports along lines parallel to the axis of the skew and not along lines normal to the two supports.

For vertical loads unsymmetrically spaced about the longitudinal axis of any arch, the stress distribution in the arch ring is extremely complex and, even for the simple case of the right arch, no satisfactory method of treatment of such eccentric loading has been developed to the speaker's knowledge; the resulting stresses can only be determined, and even then approximately, in the manner outlined in the paper.

Taking all facts into consideration, it seems clear to the speaker that as long as the loading in any skew arch is essentially vertical, the approximate theory in which laminas parallel to the longitudinal axis of the arch are investigated, can be used without introducing serious errors.

The stress distribution is entirely different when the loading is not essentially vertical, as in spandrel filled arches. In analyzing such cases, an analysis similar to that made by Professor Rathbun must be used. For instance, the

* *Proceedings*, Am. Soc. C. E., February, 1924, p. 160.

† *Loc. cit.*, p. 140.

‡ *Beton und Eisen*, 1911.

horizontal forces developed by the earth pressure in spandrel filled arches set up stress conditions in a skew arch which do not exist in the right arch.

The forces, P_s, in Fig. 11 (*a*) and (*b*), represent the horizontal components of the earth pressure against the spandrel walls, and the forces, P_a, represent the total horizontal earth pressure against the abutment and each half of the arch barrel. In the right arch, all these forces are directly opposed to each other and have no moment about a vertical axis; whereas, in the skew arch, a torsional moment is set up about a vertical axis. It is this torsional moment which modifies the stress distribution in the skew arch and develops the heavy compressive stresses at the haunches, adjacent to the obtuse corners, *A* and *C*.

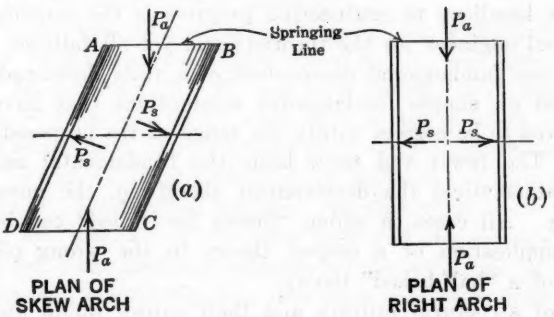


FIG. 11.

The stress distribution due to the torsional moment is extremely complex; it depends not only on the torsional moment, but also on the shape of the arch and the rigidity of the abutments. If the abutments are rigid and unyielding, the most highly stressed sections of the arch barrel will be those shown shaded in Fig. 11 (*a*). On the other hand, if the abutments are slightly displaced or rotated as a result of the torsional moment, the highest stresses in the arch barrel will be developed adjacent to the springing line at the obtuse corners, *A* and *C* (Fig. 11 (*a*)).

Summarizing, the following conclusions seem obvious to the speaker:

(*a*).—Under vertical symmetrical loading, the stress analysis of a skew arch offers no greater difficulties than a right arch, and the approximate method by taking laminas parallel to the skew appears to give results sufficiently accurate for all practical purposes.

(*b*).—Under vertical loads, unsymmetrical with respect to the longitudinal axis of the arch, the stress analysis—both of the skew and right arch—is extremely complex. The effect of these loads on the stress distribution is somewhat greater in the skew than in the right arch. For this condition of loading, a method such as that developed by Professor Rathbun must be used.

(*c*).—In spandrel filled arches, the horizontal earth pressures against the spandrel walls, arch barrel, and abutments set up a torsional moment and stress conditions in a skew arch which are not developed in the right arch. The failure to allow for resulting large torsional moment about the vertical axis is largely responsible for the many unsatisfactory skew arches recorded in engineering literature.

A. G. HAYDEN,* ESQ.—The importance of Professor Rathbun's achievement can hardly be over-estimated; this is also true of Professor Beyer's illuminating discussion of the paper as a whole and his examination of the effect of the author's approximations, which tends to inspire confidence in his conclusions.

The mathematical investigator has never received his due, because few realize the extent to which all modern science has become mathematical. The first random discoveries or chance achievements in any branch of science are spectacular and appeal to the popular imagination. Science does not progress far by hit-and-miss methods, but if development proceeds, it is only by the work of the mathematician and the patient scientific investigator.

One great handicap to engineering progress is the contempt of the self-styled practical engineer for the theorist; and yet all failures of design may be traced to one fundamental cause—lack of a fully developed mathematical analysis based on simple fundamental assumptions that have been experimentally proved to be correct within the range of the proposed application of the theory. The fewer and more basic the fundamental assumptions and the more mathematical the development therefrom, the more reliable will be the theory. All cases in which "theory has failed" can be shown to be due to the application of a correct theory in the wrong place, or to the development of a "half-baked" theory.

A study of structural failures and their causes would awaken the profession at least to a realization of the importance of such a paper as Professor Rathbun has presented. Only a small proportion of structural failures comes to the general notice, perhaps because it is just as human for the engineer to banish the memory of his failures as it is for the weather prognosticator to ignore those exceptions that belie his favorite superstition. Nevertheless, engineers as a class are the least subject to this weakness, and their failures, relative to their successes, are astonishingly few. Much more could be learned from the mistakes that are made, however, and the results would be of enormous economic value.

Competent structural designers have long felt the need of a properly developed theory of the skew arch, realizing that a discrepancy exists between the theory of the right arch and its application to the skew arch. The following examples will illustrate this statement:

(*a*).—Charles W. Comstock, M. Am. Soc. C. E., abandoned a solid skew-arch design for the Bannock Street Bridge, built in 1908 over Cherry Creek, Denver, Colo., because he had little confidence in the application of the formulas for the right arch to a structure of such extreme skew (54°) and would not risk building a structure for which no rational theory existed. A ribbed arch design with square abutting hinges at the ends was finally adopted. The bridge has a span of about 135 ft., a rise of 13 ft., and carries a 36-ft. roadway and two 8-ft. sidewalks.

(*b*).—W. Nakonz, the designer of the reinforced concrete bridge over the Berlin-Spandau Canal, in Berlin, Germany, felt the lack of a correct

* Senior Asst. Engr., Bronx Parkway Comm., Bronxville, N. Y.

theory of the fixed skew arch, and adopted a three-hinged structure. The hinges were of cast steel with hinge lines parallel to the abutments. Dowels were used in the hinges to prevent slipping along the joints, from the force due to the skew. Right arches with three hinges are statically determinate, but skew arches, even with three hinges, are statically indeterminate and this bridge was so designed, account being taken of the horizontal and vertical torsional moments at the crown and the excess thrusts at the obtuse angles of the arch barrel, all of which do not exist in right arches. The hinges were used to eliminate part of the indeterminateness and make a solution possible. This bridge, built in 1911, has two spans, one 158 ft. long, with a rise of 16.4 ft., and a skew of 35°, and the other 148 ft. long, with a rise of 14.7 ft. and a skew of 22 degrees.

(*c*).—The reinforced concrete arch bridge of the Wabash Railroad Company, over the Sangamon River, near Decatur, Ill., was designed by A. O. Cunningham, M. Am. Soc., C. E. and built in 1907. It consists of four spans, each 59 ft. long, and 29 ft. wide, with a rise of 30 ft. and a skew of 45 degrees. The designer of this bridge built a solid skew-arch bridge without adopting hinges or other expedient to make the design susceptible to known methods of mathematical analysis. At that time, there was no theory for the design and analysis of the skew arch and the stresses could not be properly analyzed. The designer safeguarded the structure by using a large factor of safety and reinforcing the arch ring heavily in both directions of both faces. The skewbacks of both the piers and abutments were saw-toothed so that, although the arch barrels were monolithic, the "assumed elements butted square into seats perpendicular to the assumed lines of thrust".

(*d*).—A few skew arches have been built with a number of longitudinal construction joints in the barrel (parallel with the skew), so that the contiguous elements were actually independent of each other.

This timidity on the part of thoughtful designers was justified. Indeed, there are instances of complete failure with loss of life which can be explained only on the basis of the real existence of the forces revealed mathematically by the author, which were not taken account of in the design. Unfortunately, many data collected by the speaker on skew arch failures, in order to sustain his position in a past controversy, have been lost and only a few examples can be recalled.

One failure in Bendigo, Australia, entailing loss of life, was due to excessive compression in the region of the obtuse angles. Professors Rathbun and Beyer showed by mathematical processes that such a condition exists in the skew arch.

One failure of a skew arch in Canada was due to an actual rotation of the arch ring on its abutments, the shifting of the ring at each abutment being toward the obtuse angle of the abutment at the acute angle of the ring. The existence of such rotational forces was revealed by the author in his mathematical exposition and also by Professor Beyer in his discussion. The arch was narrow, and the skew such that a line perpendicular to the axis of the barrel could be drawn between and entirely beyond the ends of the abutments.

Another skew-arch failure occurred several years ago in a rather peculiar manner which seemed to indicate that the cause was due to excessive compression near the obtuse angle of the arch ring combined with rotation. As the speaker recalls, the failure was reported as a "lateral shearing" of the arch ring above the skewback so that the longitudinal reinforcing rods were kinked laterally out of line across the crack. Probably the compression failure was primary, weakening the arch ring so as to invite rotation as a secondary effect.

During 1923, one skew span of the Schenectady-Scotia reinforced concrete bridge over the Mohawk River collapsed before the falsework was removed, five men losing their lives. A complete report of this failure has not been published, but it is probable that the lateral displacement of the piles supporting the falsework trusses was due to rotational forces in the several arch ribs, which were not taken account of in the design either of the bridge itself or of the falsework.

An interesting occurrence was observed during the construction of Bridge No. 1, on the Bronx River Parkway, designed by Guy Vroman, M. Am. Soc. C. E., and constructed under the supervision of the engineers of the Bronx Parkway Commission. This bridge, built in 1920, consists of two parallel reinforced concrete arches, one for north-bound and the other for south-bound traffic. Each arch is 37 ft. wide perpendicular to the center line of the roadway, with a span of 70 ft., parallel to the roadway, and a rise of $13\frac{1}{2}$ ft., and a skew of 54 degrees. The contractor submitted a design for the falsework, consisting of several lines of timber trusses parallel with the proposed roadway. As one condition of approval, the speaker ordered the bracing between the trusses to be arranged so as to resist the forces due to the tendency of the trusses to rotate under the weight of the concrete in the arch ring. The contractor then, on his own initiative, added bracing in an opposite direction, although advised of its redundancy. During the casting of the arch ring, the bracing put in by order of the speaker was taut under stress, whereas that added by the contractor was slack and could be easily shaken by hand. Thus, the tendency of the trusses to rotate in such manner that their ends would shift toward the obtuse angles of the abutments was decidedly apparent, although properly resisted by the bracing. The abutments of this bridge were saw-toothed so that failure, after removal of the falsework, by slipping of the ring along the skewbacks due to rotational forces, was prevented.

Professor Rathbun is the first to succeed in fully developing a theory and method of analysis for the skew arch. The first attempt to discover the general principles of the stability of a skew arch was made by a German engineer, Dr. E. Fischer,* but his formulas do not take account of all the elements entering into the problem, and he makes no attempt to analyze the stresses in the arch ring itself. The paper was "to be continued", but further installments do not appear in subsequent issues of *Beton und Eisen.*

---

* "Beitrag zur Berechnung und zum Bau schiefer gewölbter Brücken," *Beton und Eisen*, November 8, 1911.

The U. S. Bureau of Public Roads is conducting an elaborate series of tests on models of concrete skew arches, in order to determine experimentally the action of such structures. The results of these tests should be extremely valuable in corroborating the conclusions reached by Professor Rathbun. They will not assist, however, in the quantitative analysis of actual structures, which, of necessity, must be mathematical. For proving fundamental assumptions, or for revealing phenomena which are not apparent at the outset and which must form the basis of a rational theory, experimental investigation is essential; but the "carrying-on" must be assumed by the mathematician.

E. H. HARDER,* Assoc. M. Am. Soc. C. E. (by letter).†—The author deserves great commendation for his earnest endeavors to place the design of the concrete skew arch on a more substantial basis; yet, having given this same problem considerable study, the writer feels that the battle has just begun. This paper should be fully discussed and, out of the interest aroused, some practical solution should be evolved, that will enable the designer to restore faith in the skew arch.

Owing to the complications of the problem, it is doubtful whether the desired practical method of analysis will be simple enough to meet the needs of the practicing engineer. Future investigations of the skew arch will probably lead to a series of "don'ts" rather than to a rigid analysis and, therefore, the design of such arches will be largely empirical.

The author has stated that the literature on this subject is nil, or, at least, that he had been unable to locate such literature. The writer likewise has searched the indexes of American and European engineering literature and will give a brief review of several of the articles found.

Probably the first reference to the subject of the concrete skew arch is to be found in a paper‡ by the late W. C. Kernot, M. Am. Soc. C. E., who was Professor of Engineering at the University of Melbourne, Melbourne, Victoria, Australia. Professor Kernot had been engaged to defend a contractor who had constructed a skew arch at Bendigo that collapsed on May 13, 1902. He did not undertake a mathematical analysis, but had to content himself by experimenting with models. Some of these experiments were truly ingenious, and from them he formulated the following rules (angle of skew, 90°, signifying a square bridge):

(1) Skew above 80°—design as at present (that is, same as a square bridge).

(2) Skew 80° to 70°—the acute angle of each abutment and pier should be replaced by a right angle.

(3) Skew 70° to 60°—increase the thickness of the arch by one-fourth at the skewbacks near the acute angles of the abutment, such increased thickness to diminish in all directions until it vanishes at a distance equal to one-fourth of the skew span. Replace acute angles of piers and abutments by right angles, and increase the thicknesses at these points by one-fourth, such increased thickness to diminish in the same manner as that of the arch.

---

* Structural Engr., Concrete Steel Eng. Co., New York, N. Y.

† Received by the Secretary, March 17, 1924.

‡ *Engineering News*, June 11. 1903, pp. 529-530.

(4) Skew 60° to 50°—adopt a treatment similar to Rule (3), except that the increased thickness of arches, piers, and abutments should be one-third.

(5) Skew 50° to 40°—adopt a similar treatment, except that the increased thickness in each case should be one-half.

The experimentation on which this semi-empiric method is based disclosed the fact that the thrust of a skew arch is not parallel to the arch faces but, because of lateral shear, lateral bending, and torsion, is concentrated near the acute angles of the abutments.

The next reference is a paper* by Dr. E. Fischer which was discussed† by Clyde T. Morris, M. Am. Soc. C. E.

Foreign literature disclosed two important articles, both in the German language, the first by C. Busemann,‡ and the second by Dr. H. Marcus.§ Of these, the latter is by far the more important. Dr. Marcus first treats the general case of a beam curved in two or three directions, that is, a space curve, and, finally, applies it to the analysis of a skew arch. The present discussion will be confined largely to a comparison of his method and that of Professor Rathbun.

From a mathematical viewpoint the author's paper is somewhat unsatisfactory, because of the assumed positions of the axes. Experimentation, alone, will disclose the error involved. Not only can one take exception to the x, y, and z-axes, but the u, v, and z-axes of any point, P, must likewise be viewed with suspicion, because they are disjointed and bear no direct relation to the other axes. In the analysis of a symmetrical right arch, the principal axes are vertical and horizontal, the corresponding u and v-axes of any point, P, are inclined to the principal axes, and the functions involved, with respect to the angle of inclination, are $\sin \phi$ and $\cos \phi$.

As the axis of a skew arch is a space curve, because two dimensions do not completely describe it, at any point the v-axis must be tangent to the arch axis, the u-axis radial and the z-axis perpendicular to the other two or conjugate to the u-axis. The cosines of the nine angles which the v, u, and z-axes make with the positive branches of the x, y, and z-axes of the references in the general analysis of Dr. Marcus, are shown in Table 7 and may be substantiated by referring to any work on Analytical Geometry.

TABLE 7.

	x-axis.	y-axis.	z-axis.
v-axis....................	a_1	b_1	c_1
u-axis....................	a_2	b_2	c_2
z-axis....................	a_3	b_3	c_3

* *Beton und Eisen*, 1911, p. 391.

† In "Thrust of Skew Barrel Arch Measured in Laboratory Model", *Engineering News-Record*, Vol. 88, No. 16 (April 20, 1922), and in subsequent letters by other contributors.

‡ "Untersuchungen über die Kraftrichtung im Schiefen Gewölbe," Berlin, 1910.

§ "Der Doppelt gekrümmte Träger und das Schiefe Gewölbe."

The determination of these angular functions is a simple one if the equation of the arch axis is known:

$$a_1 = b_2\, c_3 - b_3\, c_2;\; b_1 = c_2\, a_3 - c_3\, a_2;\; c_1 = a_2\, b_3 - a_3\, b_2.$$

$$a_2 = \frac{d\,x}{d\,s};\; b_2 = \frac{d\,y}{d\,s};\; c_2 = \frac{d\,z}{d\,s};$$

$$a_3 = \frac{A}{D};\quad b_3 = \frac{B}{D};\quad c_3 = \frac{C}{D};$$

in which,

$$d\,s = \sqrt{d\,x^2 + d\,y^2 + d\,z^2}$$
$$A = d\,y\,d^2\,z - d\,z\,d^2\,y$$
$$B = d\,z\,d^2\,x - d\,x\,d^2\,z$$
$$C = d\,x\,d^2\,y - d\,y\,d^2\,x$$
$$D = \sqrt{A^2 + B^2 + C^2}$$

Referring to the statement that the axes assumed by the author are disjointed, the foregoing device gives the cosines of angles which his *v*, *u*, and *z*-axes make with the positive branches of the *x*, *y*, and *z*-axes of reference at the crown of the arch, as indicated in Table 8.

TABLE 8.

	x-axis.	*y*-axis.	*z*-axis.
v-axis..........................	$a_1 = \cos\phi$	$b_1 = -\sin\phi$	$c_1 = 0$
u-axis..........................	$a_2 = \sin\phi$	$b_2 = \cos\phi$	$c_2 = 0$
z-axis..........................	$a_3 = 0$	$b_3 = 0$	$c_3 = 1$

From Table 8 it should be apparent that there is no continuity in the author's arch axis as referred to his system of co-ordinates. This is tantamount to attempting to analyze a right arch by making each *v* and *u*-axis of the individual arch elements horizontal and vertical, respectively.

This leads to another consideration which is also of great importance. It is unfortunate that the author did not conclude his paper by showing his treatment of the arch after finding the six unknowns. Does he find the stresses on the diagonal sections parallel to the springing line? If so, he presents an anomaly which is obviously wrong. Referring to Fig. 12, it is apparent that, at the ends of his diagonal section, there exists a condition in which a unit thrust, for example, acts on a feather edge while directly opposite, a like unit thrust acts on solid concrete, that is, a unit thrust acting on one side of a section is opposed not by an equal unit thrust, but by zero.

To overcome this serious objection, Dr. Marcus takes a section, *A B* (Fig. 13), along the center line of the arch, the shaded triangles at *A* and *B* being assumed as rigid and inelastic parts of the abutments.

Cross-sections at right angles to the side faces of the arch, appear as a series of trapeziums with the long sides curved (Fig. 14). The locus of the centers

of gravity of these sections forms a curve lying half way between the extrados and intrados in elevation (Fig. 15 (*a*)), while in plan (Fig. 15 (*b*)), it is S-shaped, because, as the sections approach the abutment, the centers of gravity move closer and closer to the obtuse angle between the springing line and the face of the arch. Thus, the neutral axis of a skew arch must be a space curve and have all the geometrical properties of such a curve, as previously indicated.

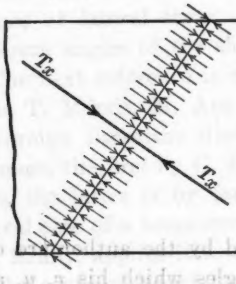


FIG. 12.

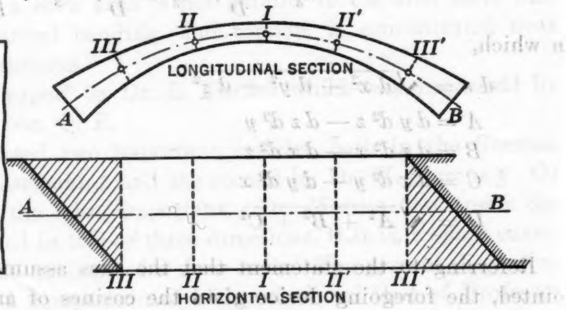


FIG. 13.

Here, however, Dr. Marcus introduces two approximations. As the divergence of the true neutral axis from the vertical plane through the center line of the bridge is small (Fig. 15 (*b*)), he assumes that it lies in this plane. Consequently, $z = 0$ and, therefore, $c_1 = 0$. His second assumption considers that each section is a parallelogram and that u, the radial axis, bisects the section as shown in Fig. 16. The z-axis at this point would be conjugate to the u-axis but, owing to the labor of computation, this axis is assumed to bisect the parallelogram in its long direction and to make an angle, ψ, with the horizontal.

FIG. 14.

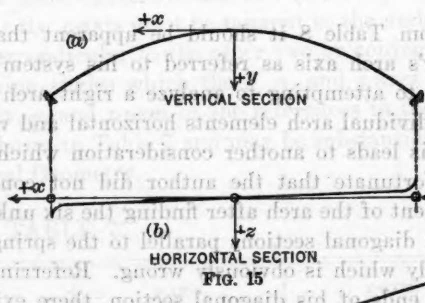


FIG. 15

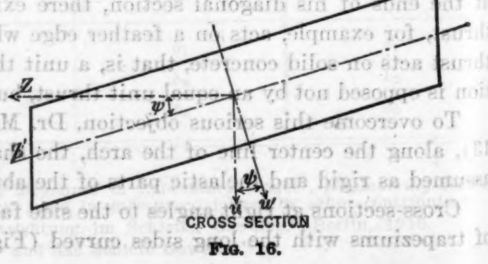


FIG. 16.

The cosines of the angles which the v, u, and z-axes make with the positive branches of the x, y, and z-axes of reference for Dr. Marcus' skew arch axis, are as given in Table 9.

TABLE 9.

	x-axis.	y-axis.	z-axis.
v-axis..........................	$a_1 = \cos \phi$	$b_1 = \sin \phi$	$c_1 = 0$
u-axis..........................	$a_2 = -\cos \psi \sin \phi$	$b_2 = \cos \psi \cos \phi$	$c_2 = -\sin \psi$
z-axis..........................	$a_3 = -\sin \psi \sin \phi$	$b_3 = \sin \psi \cos \phi$	$c_3 = \cos \psi$

The previous conception of the skew arch is now somewhat changed; instead of a slice of a right arch of great width cut by two diagonal vertical planes, it resembles a curved spring with square ends that are twisted with respect to one another and attached to the rigid triangular projections of the abutments (Fig. 13).

Dr. Marcus also gives a ready method of finding the necessary moments of inertia for each section. These data are then substituted in the general analysis for a doubly curved beam from which six simultaneous equations are obtained similar to those of the author. In fact, the author's equations can be obtained from the general equations of Dr. Marcus' paper, except the various loading factors which would necessarily have to be developed to suit the new conditions.

To illustrate his method of analysis, Dr. Marcus computes the indeterminate reactions for the arch shown in Fig. 17 for a load, P, in the center of the crown, as follows: $T_x = +1.294\,P$; $T_y = 0.0$.; $T_z = +0.046\,P$; $M_x = -0.071\,P$; $M_y = 0.0$; and, $M_z = +1.576\,P$.

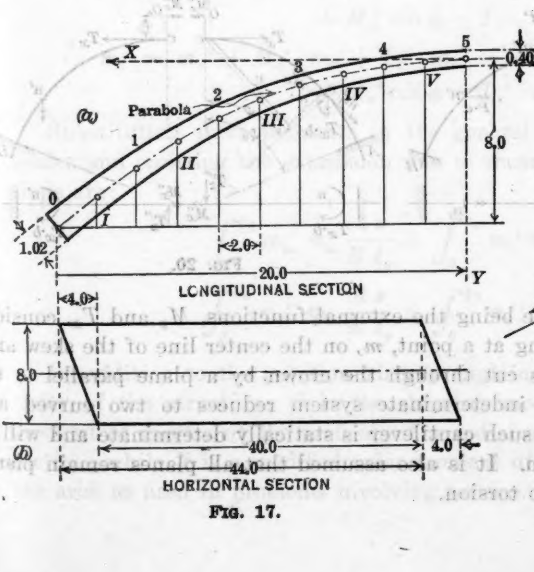


FIG. 17.

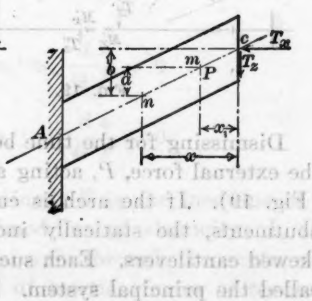


FIG. 18.

This method would serve very satisfactorily for slender rib arches or for barrel arches in which the angle of skew was nearly 90 degrees. By an inspection of Fig. 13, it is apparent that for wide ribs and heavy skew, the two shaded triangles would touch each other or even overlap; then, this method would be inapplicable.

It seems to the writer that the solution of the skew arch problem must be sought by other methods still undiscovered and until the time arrives when the mysteries of this type of structure are fully disclosed engineers must be content to design the few necessary skew arches by semi-empiric methods, that is, according to Professor Kernot's rules, or to some similar set of conclusions based on more modern experimentation. If, however, the designer is not satisfied to do this and wishes to appease his conscience by computing shears and torsions on diagonal sections, he can use the author's method or expand the following method devised by the writer, which will give him the same results.

One familiar with the general problem of the "double curved beam", as developed by Dr. Marcus and others, would realize that Professor Rathbun's development is the analysis of a right arch plus a third dimension added to make it complicated and that the problem can also be solved by first analyzing the skew arch as if it were a right arch and then computing torsion and transverse shears as secondary forces. To obtain the primary forces, the skew arch (Fig. 18) is first projected on a vertical plane at right angles to the springing line (Fig. 19) and the secondary forces are obtained according to the following method which will be developed for the simplest case of a symmetrical arch with symmetrical loading. For such a case, the vertical crown shear is zero, the transverse crown bending moment is zero, and T_x and M_z are the same as for the right arch. This method could be further expanded to include unsymmetrical arches and unsymmetrical loading.

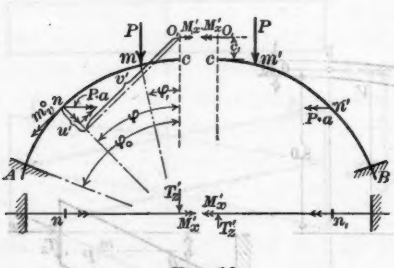


FIG. 19.

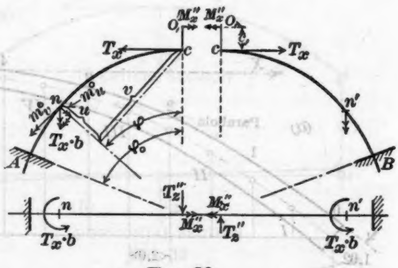


FIG. 20.

Dismissing for the time being the external functions, M_z and T_x, consider the external force, P, acting at a point, m, on the center line of the skew arch (Fig. 19). If the arch is cut through the crown by a plane parallel to the abutments, the statically indeterminate system reduces to two curved and skewed cantilevers. Each such cantilever is statically determinate and will be called the principal system. It is also assumed that all planes remain planes after they are subjected to torsion.

The force, P, causes a twisting moment in a vertical plane parallel to the abutment about Point n equal to $P \,.\, a$, or $P\,(x - x_1) \tan \epsilon$ (Fig. 18). For the purpose of analyzing the effect of such twisting moments, consider the right arch as a principal system on which variable twisting moments in vertical planes parallel to the abutments act.

These moments are zero at P and increase toward the abutment. At Point n, the twisting moment, $P \,.\, a$, is shown as a vector quantity. The double-headed arrow distinguishes such vectors representing moments, from the ordinary single-headed arrow used to represent forces. Looking at the double head of such a vector in the direction of the shaft of the arrow, any moment rotating in a clockwise direction about the shaft is positive.

The moment, $P \,.\, a$, may be resolved into component moments, one a bending moment, bending the radial section at n in a transverse direction, the other a torsional bending moment acting in the plane of the radial section. Here, the writer must vary the notation of the paper and adopt his own. The former is $m_u^\circ = P \,.\, a \sin \phi$; the latter is $m_v^\circ = -P \,.\, a \cos \phi$.

Similar moments act at symmetrical points like n' in the right half of the arch; but when viewed in elevation the direction of the moments acting on the right half appears opposite to those on the left.

These moments cause two unknown reactions at a point, O_1, to be determined later, which will be the origin of co-ordinates just as some similar point below the crown is adopted for a right arch. The unknown reactions are a torsional moment, M_x', and a horizontal transverse shear, T_z'. (Fig. 19.)

Making $M_x' = 1$, its corresponding components at n will be: $m_u' = \sin \phi$, and $m_v' = -\cos \phi$; and if $T_z' = 1$, its corresponding moment components at n will be: $m_u'' = -v'$, and $m_v'' = -u'$.

Thus, the total component moments due to external forces at n and reactions at O_1 will be:

$$m_u = m_u^\circ + M_x' \,.\, m_u' + T_z' \,.\, m_u'' = P\,(x - x_1) \tan \epsilon \,.\, \sin \phi + M_x' \sin \phi - T_z' \,.\, v'.$$

$$m_v = m_v^\circ + M_x' \,.\, m_v' + T_z' \,.\, m_v'' = -P\,(x - x_1) \tan \epsilon \,.\, \cos \phi - M_x' \cos \phi - T_z' \,.\, u'.$$

Substituting these moments in the general work equations for isotropic bodies and omitting the expression due to shearing deformations, these equations are:

$$\int_0^{\phi_0} m_u' \, m_u \frac{d\,s}{E\,I_y} + \int_0^{\phi_0} m_v' \, m_v \frac{d\,s}{G\,F} = 0$$

$$\int_0^{\phi_0} m_u'' \, m_u \frac{d\,s}{E\,I_y} + \int_0^{\phi_0} m_v'' \, m_v \frac{d\,s}{G\,F} = 0$$

In addition to the notation already explained, E is the modulus of elasticity of the structure; G is the shearing modulus of elasticity; I_y is the moment of inertia of the arch about a radial axis through the center of the section; while F is a modified polar moment of inertia (factor of torsion) of the arch as used in problems involving torsion.

According to Bach and Bretschneider,* this moment of inertia is:

$$F = \frac{b^3 d^3}{(b^2 + d^2)\left(3.645 - 0.06\,\frac{b}{d}\right)}\dagger$$

After substituting the expressions for the various moments involved, the general work equations become:

$$\int_{\phi_1}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(\sin^2\phi + \nu\cos^2\phi)\,d\,w + M_x' \int_0^{\phi_0} (\sin^2\phi$$
$$+ \nu\cos^2\phi)\,d\,w - T_z' \int_0^{\phi_0} (v'\sin\phi - \nu\,.\,u'\cos\phi)\,d\,w = 0$$

$$-\int_{\phi_1}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(v'\sin\phi - \nu\,.\,u'\cos\phi)\,d\,w - M_x' \int_0^{\phi_0} (v'\sin\phi$$
$$- \nu\,u'\cos\phi)\,d\,w + T_z' \int_0^{\phi_0} (v'^2 + \nu\,u'^2)\,d\,w = 0$$

Here,

$$d\,w = \frac{d\,s}{I_y},\ \text{and}\ \nu = \frac{E\,I_y}{G\,F} = \frac{1}{12}\left(1 + \frac{b^2}{d^2}\right)\left(3.645 - 0.06\,\frac{b}{d}\right)\frac{E}{G}$$

In these expressions, the integral, $\int_0^{\phi_0} (v'\sin\phi - \nu\,u'\cos\phi)\,d\,w$, appears twice, as a factor of T_z' and also of M_x'. If these factors could be made zero, the equations would be greatly simplified.

As $v' = v + c_1\,.\sin\phi$ and $u' = u - c_1\,.\cos\phi$, the integral mentioned would become:

$$\int_0^{\phi_0} (v\sin\phi - \nu\,u\cos\phi)\,d\,w + c_1 \int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w$$

Placing this expression equal to zero and solving for c_1, we have:

$$c_1 = -\frac{\int_0^{\phi_0} (v\sin\phi - \nu\,u\cos\phi)\,d\,w}{\int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w}$$

If c_1 is plus, it lies above the crown of the arch and if minus, below.

This gives quickly and simply expressions for M_x' and T_z' which act at Point O_1:

$$M_x' = -\frac{\int_{}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(\sin^2\phi + \nu\cos^2)\,d\,w}{\int_0^{\phi_0} (\sin^2\phi + \nu\cos^2\phi)\,d\,w}$$

$$T_z' = \frac{\int_{\phi_1}^{\phi_0} P\,(x - x_1)\tan\varepsilon\,(v'\sin\phi - \nu\,u'\cos\phi)\,d\,w}{\int_0^{\phi_0} (v'^2 + \nu\,u'^2)\,d\,w}$$

* "Versuche über die Verdrehung von Stäben mit rechteckigem Querschnitt", *Mitteilungen über Forschungsarbeiten,* Heft 21.

† Or use the author's Equation (2), *Proceedings,* Am. Soc. C. E., February, 1924, p. 140.

As the summations involving $P\ (x - x_1)\tan\varepsilon$ would be tedious, it would be far simpler to plot influence lines for the condition, $P = 1$, and then load these influence lines with the actual weights of the arch and fill for dead load.

The equations given previously are correct if equal loads act at symmetrical points with respect to the crown of a symmetrical arch. Before plotting influence lines for a moving load of $P = 1$, it is necessary to consider another factor. These formulas were derived for the case of equal moments at symmetrically located points, but having opposite rotational directions. Considering the case of two equal moments symmetrically located, but having the same direction when viewed in elevation, and proceeding as before, it is found that there is one unknown reaction at the crown, a bending moment acting at right angles to the plane of the crown section and expressed mathematically,

$$M_v' = -\frac{\int_{\phi_1}^{\phi_0} P\ (x - x_1)\tan\varepsilon\ (\sin\phi\cos\phi)\ (1 - \nu)\ d\,w}{\int_0^{\phi_0} (\cos^2\phi + \nu\sin^2\phi)\ d\,w}$$

Adding the two cases of loading, would give $2\,P\,.\,a$ on the left half of the arch and zero moments on the right half, as here the moments are equal, but opposite in sign.

Hence, for moments equal to $2\,P\,.\,a$ on the left half of the arch, the three unknowns are M_x', T_z' and M_v'. It follows, therefore, that, to plot the influence lines for M_x' and T_z' for $1\,P\,.\,a$ on the left half of the arch, the expressions given for M_x' and T_z' must be divided by 2.

As summations instead of integrations are required, the arch must be cut into finite sections, and therefore, $d\,w$ becomes $\Delta\,w = \dfrac{\Delta\,s}{I_y}$.

The equations given previously may be further simplified if,

$$(\cos^2\phi + \nu\sin^2\phi)\,\Delta\,w = w_1$$
$$(\sin^2\phi + \nu\cos^2\phi)\,\Delta\,w = w_2$$
$$(1 - \nu)\sin\phi\cos\phi\,\Delta\,w = w_3$$

Furthermore, as,

$$v' = v + c_1\,.\sin\phi = x\cos\phi + (y + c_1)\sin\phi$$
$$u' = u - c_1\,.\cos\phi = x\sin\phi - (y + c_1)\cos\phi$$

the numerator of the equation for T_z' becomes:

$$\int_{\phi_1}^{\phi_0} P\ (x - x_1)\tan\varepsilon\ [x\ (1 - \nu)\sin\phi\cos\phi + (y + c_1)(\sin^2\phi + \nu\cos^2\phi)]\ d\,w$$

or,

$$\sum_{\phi_1}^{\phi_0} P\ (x - x_1)\tan\varepsilon\ [x\,w_3 + (y + c_1)\,w_2]$$

and the equation for T_z', for a load, $P = 1$, on the left half of the arch only, becomes:

$$T_z' = \frac{\tan\varepsilon}{2\,N_3}\sum_{\phi_1}^{\phi_0} (x - x_1)\,[x\,w_3 + (y + c_1)\,w_2]$$

in which,

$$2\,N_3 = 2 \sum_{0}^{\phi_0} (v'^2 + \nu\, u'^2)\, \Delta\, w$$

Likewise,

$$M_x' = -\frac{\tan \varepsilon}{2\,N_2} \sum_{\phi_1}^{\phi_0} (x - x_1)\, w_2$$

in which,

$$2\,N_2 = 2 \sum_{0}^{\phi_0} w_2$$

Graphically, the influence line for M_x' may be drawn by letting the elastic weights, w_2, act vertically on half the arch. Then, any ordinate, η, will equal $\sum_{\phi_1}^{\phi_0} (x - x_1)\, w_2$ when multiplied by the pole distance, H. Having multiplied the ordinates of this influence line by the corresponding P loads, add these products, and then multiply by $\dfrac{H \tan \varepsilon}{2\,N_2}$. (Fig. 21.)

The influence line for T_z' may be found in the same manner. First, let the elastic weights, w_3, act vertically and draw the string polygon, extending each side until it intersects the Y-axis. Then, let the elastic weights, w_2, act horizontally and draw the polygon for these weights in the horizontal position. Prolong each side of the polygon until it intersects the horizontal X-axis. Then, take each pair of corresponding intercepts, $\Delta\, x_1$ and $\Delta\, y_1$, and add algebraically. The sign of these intercepts is determined by the sign of w_3 and w_2. Use these summations to form a force polygon. If for both these operations the same pole distance, H, was used, then the ordinates, η, are multiplied by the corresponding load, P, then by $H \,.\, \tan \epsilon$, and, finally, divided by $2\,N_3$.

The force, T_x, acting on half the skew arch, causes a moment about n equal to $T_x \,.\, b$, or $T_x \,.\, x \tan \epsilon$. These moments lie in a horizontal plane and rotate about a vertical axis. Vectorially, they are shown in Fig. 20. The T_x acting on the right half of the arch causes moments about points on the axis in the same direction as those caused by T_x on the left half of the arch.

This case, therefore, comprises two equal moments symmetrically placed with respect to the center of the arch, acting in horizontal planes and having the same rotational direction. There are two statically indeterminate reactions, M_x'' and T_z'', both acting at O_1.

$$m_u^\circ = T_x \,.\, x \tan \epsilon \cos \phi \qquad\qquad m_v' = -\cos \phi$$
$$m_v^\circ = T_x \,.\, x \tan \epsilon \,.\, \sin \phi \qquad\qquad m_u'' = -v'$$
$$m_u' = \sin \phi \qquad\qquad m_v'' = -u'$$

In the statically indeterminate system:

$$m_u = T_x \,.\, x \tan \epsilon \,.\, \cos \phi + M_x'' \sin \phi - T_z'' \,.\, v'$$
$$m_v = T_x \,.\, x \tan \epsilon \,.\, \sin \phi - M_x'' \cos \phi - T_z'' \,.\, u'$$

The two elastic equations after multiplying by E are:

$$\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,.\, \sin \phi \cos \phi \,(d\,w - d\,w_1) + M_x'' \int_0^{\phi_0} (\sin^2 \phi \, d\,w + \cos^2 \phi \, d\,w_1)$$

$$- T_z'' \int_0^{\phi_0} (v' \sin \phi \, d\,w - u' \cos \phi \, d\,w_1) = 0$$

$$- \int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,(v' \cos \phi \, d\,w + u' \sin \phi \, d\,w_1) - M_x'' \int_0^{\phi_0} (v' \sin \phi \, d\,w$$

$$- u' \cos \phi \, d\,w_1) + T_z'' \int_0^{\phi_0} (v'^2 \, d\,w + u'^2 \, d\,w_1) = 0$$

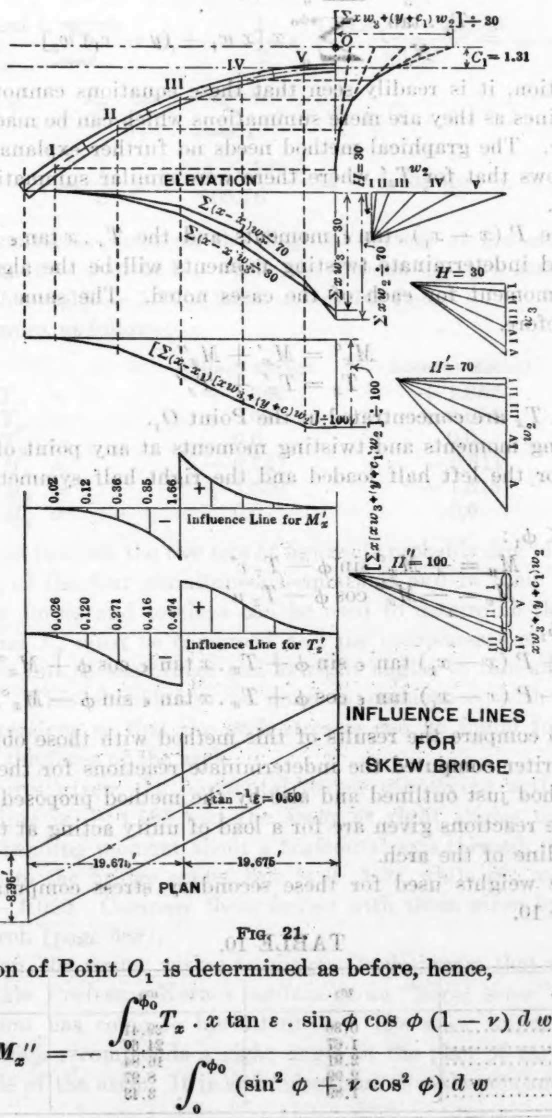


FIG. 21.

The position of Point O_1 is determined as before, hence,

$$M_x'' = - \frac{\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,.\, \sin \phi \cos \phi \,(1 - \nu)\, d\,w}{\int_0^{\phi_0} (\sin^2 \phi + \nu \cos^2 \phi)\, d\,w}$$

$$T_z'' = + \frac{\int_0^{\phi_0} T_x \,.\, x \tan \varepsilon \,(v' \cos \phi + \nu \, u' \sin \phi) \, d\, w}{\int_0^{\phi_0} (v'^2 + \nu \, u'^2) \, d\, w}$$

or, simplifying as before:

$$M_x'' = - \frac{T_x \,.\, \tan \varepsilon}{N_2} \sum_0^{\phi_0} x \,(1 - \nu) \sin \phi \cos \phi \, \Delta\, w = - \frac{T_x \,.\, \tan \varepsilon}{N_2} \sum_0^{\phi_0} x \, w_3$$

$$T_x'' = + \frac{T_x \,.\, \tan \varepsilon}{N_3} \sum_0^{\phi_0} x \,(v' \cos \phi + \nu \, u' \sin \phi) \, \Delta\, w$$

$$= \frac{T_x \,.\, \tan \varepsilon}{N_3} \sum_0^{\phi_0} x \,[x \, w_1 + (y + c_1) \, w_3]$$

By inspection, it is readily seen that these equations cannot be expressed by influence lines as they are mere summations which can be made analytically or graphically. The graphical method needs no further explanation; the construction follows that for T_z' where there were similar summations along the X and Y-axes.

Due to the $P\,(x - x_1)\,.\,\tan \epsilon$ moments and the $T_x\,.\,x \tan \epsilon$ moments, the total combined indeterminate twisting moments will be the algebraic sum of the twisting moment for each of the cases noted. The same is true of the shears. Therefore.

$$M_x^\circ = M_x' + M_x''$$
$$T_z = T_z' + T_z''$$

Both M_x° and T_x are concentrated at the Point O_1.

The bending moments and twisting moments at any point of the left half of the arch for the left half loaded and the right half symmetrically loaded will be:

For $\phi < \phi_1$:

$$M_u = + M_x^\circ \sin \phi - T_z \, v'$$
$$M_v = - M_x^\circ \cos \phi - T_z \, u'$$

For $\phi > \phi_1$:

$$M_u = + P\,(x - x_1) \tan \epsilon \sin \phi + T_x\,.\,x \tan \epsilon \cos \phi + M_x^\circ \sin \phi - T_z \, v'$$
$$M_v = - P\,(x - x_1) \tan \epsilon \cos \phi + T_x\,.\,x \tan \epsilon \sin \phi - M_x^\circ \cos \phi - T_z \, u'$$

In order to compare the results of this method with those obtained by Dr. Marcus, the writer computed the indeterminate reactions for the arch of Fig. 17 by the method just outlined and also by the method proposed by Professor Rathbun. The reactions given are for a load of unity acting at the crown and on the center line of the arch.

The elastic weights used for these secondary stress computations are as given in Table 10.

TABLE 10.

Point.	w_1.	w_2.	w_3.
V..................................	0.36	29.41	−2.38
IV.................................	1.67	24.67	−6.07
III................................	2.91	16.84	−6.81
II.................................	2.99	8.97	−5.05
I..................................	1.89	3.48	−2.45

$$c_1 = -\frac{1}{N_2}\sum (x\,w_3 + y\,w_2) = -\frac{-109.21}{83.32} = 1.31 \text{ ft.}$$

$$M_z' = -\frac{P}{2}\frac{\tan\varepsilon}{N_2}\sum_{\phi_1}^{\phi_0}(x - x_1)\,w_2 = -\frac{1}{2}\times\frac{1}{2}\times\frac{553.7}{83.32} = -1.660 \text{ ft-lb.}$$

$$T_z' = \frac{P}{2}\frac{\tan\varepsilon}{N_3}\sum_{\phi_1}^{\phi_0}(x - x_1)\,[x\,w_3 + (y + c_1)\,w_2]$$

$$= \frac{1}{2}\times\frac{1}{2}\times\frac{-561.2}{295.76} = -0.474 \text{ lb.}$$

$$M_z'' = \frac{T_x \cdot \tan\varepsilon}{N_2}\sum_0^{\phi_0} x\,w_3 = \frac{1}{2}\times\frac{1.342\times -220.43}{83.32} = +1.775 \text{ ft-lb.}$$

$$T_z'' = \frac{T_x \cdot \tan\varepsilon}{N_3}\sum_0^{\phi_0} x\,[x\,w_1 + (y + c_1)\,w_3]$$

$$= \frac{1}{2}\times\frac{1.342\times 498.5}{295.76} = +1.132 \text{ lb.}$$

The total transverse shear on the diagonal section at the crown is equal to $+1.132 - 0.474 = +0.658$ lb., while the torsion on the same section is equal to $+1.775 - 1.660 - 1.31\times 0.658 = -0.747$ ft-lb.

The results were, as follows:

	Writer's Method.	Author's Method.
T_x =	+ 1.342	+ 1.352
T_z =	+ 0.658	+ 0.674
T_y =	0.0	0.0
M_x =	− 0.747	− 0.811
M_z =	+ 1.530	+ 1.515
M_y =	0.0	0.0

The difference between the two sets of figures is probably due to inaccuracies in computation of the four simultaneous equations and in their solution.

Before these forces and torsions can be used to determine the stresses in the arch, T_x and T_y must be combined and the components of the resultant along the center line of the bridge and at right angles to this axis must then be determined. This should also be done in connection with the bending moments and torsions so that the arch stresses can be found for sections at right angles to the faces of the arch.

For the figures given, the horizontal thrust component acting along the bridge center line is + 1.485 and the shear at right angles to this line is + 0.016. The bending moment about a horizontal axis through the crown and at right angles to the bridge center line is + 1.70, while the torsion on this section is = − 0.020. Compare these figures with those given by Dr. Marcus for the same arch (page 689).

In conclusion, the writer wishes to suggest to designers that the rules laid down by the late Professor Kernot contain more "horse sense" than all the mathematics that has come to his attention. The most important of these rules is that which recommends a right angle in the plan of the abutment at the obtuse angle of the arch. It is also important to make abutments of ample size.

It is purely a waste of time to make fancy calculations for an arch with a span of 36 ft., a rise of 14 ft., a width of 60 ft., a thickness of 14 in. at the crown and 72 in. at the skewbacks. The extreme width of 60 ft. precludes all torsion computations, because such a long section for a small depth does not follow the usual laws pertaining to torsion on which the analyses in question depend. The mathematical analyses of the skew arch mentioned in this discussion may be of considerable value in the design of skewed rib arches in which the ratio of width to depth is low; but an attempt to apply the same analyses to very wide ribs, involves assumptions which are not justifiable.

JOHN SANFORD PECK,* ASSOC. M. AM. SOC. C. E. (by letter).†—In subjecting the skew arch to a rigorous mathematical analysis, Professor Rathbun has brought to light the inherent weakness of most of the skew arches designed to date, and incidentally has provided the profession with logical explanations of the numerous failures of this type of structure as well as the basis for safe and sane design in the future.

The first point emphasized by the author, namely, that the stress does not take the shortest route to the abutment, but follows along the axis of the arch, shows among other things the fallacy of the so-called "common sense" theories. Offhand, it seems plausible that the stress would take the shortest route to the abutment, but the mathematical analysis shows conclusively that such is not the case and, therefore, common sense must give way to mathematics.

Unfortunately, the average engineer, or designing draftsman, is not sufficiently skilled in mathematics to be able to make such an analysis. The writer is of the opinion that even if the designer had the equations which Professor Rathbun had deduced for expressing the crown thrust, he would be unable to solve them after having made the proper substitutions. The criticism of the majority of engineers would probably be that this theory is too involved and technical to be practical.

Granting this to be true—an opinion in which the writer does not concur—the paper is invaluable nevertheless for it serves to point out specifically the dangers in this type of structure and thus enables the designer to take care of them.

The theory has been advanced that an approximate design consistent with good practice can be obtained by considering the arch as a number of adjacent narrow rings and by actually building it with construction joints corresponding. Each arch element is designed as a right arch. This theory can be used with safe results as proved by an arch built by the Bronx River Parkway Commission under the direction of Mr. A. G. Hayden, Chief Designing Engineer; tests on models showed the necessity of keying the abutments to take care of the tendency to slip parallel to the springing line and, further, of providing for the horizontal thrust of the earth fill against the spandrel walls.

Of course, this method is only begging the question, for the face of the keys in the abutment are perpendicular to the axis of the arch element, and the resulting condition gives a number of narrow right arches placed together

* Supt. of Constr., Allen N. Spooner & Son, Inc., New York, N. Y.

† Received by the Secretary, March 24, 1924.

to make up a structure that is a skew arch in appearance and name only. The earth pressure against the spandrel walls can be eliminated by building retaining walls at the abutments.

It would be interesting to compare the time required for the approximate design mentioned with that for the complete mathematical analysis according to Professor Rathbun's theory. The writer is of the opinion that the balance would be in favor of the complete—and, incidentally, safe—mathematical analysis and would prove conclusively that Professor Rathbun is a decidedly practical theorist.

J. CHARLES RATHBUN,* M. AM. SOC. C. E. (by letter).†—That there has been a demand for the solution of the problem involved in skew arch design has been brought out in a gratifying manner by Mr. Hayden,‡ while the other discussions show that it has received considerable attention from the profession.

In preparing this paper, the writer adopted a treatment that would appeal to the practical engineer rather than to the theorist and that could be read by the designing draftsman as well as the engineer. This required the incorporation of a large amount of algebra which has probably, in a measure, defeated its own purpose. The nature of the problem is such that the equations deduced are lengthy but, fortunately, not difficult to utilize.

To analyze a given ring for definite loads a designing draftsman simply substitutes figures from his layout for the letters in the formulas. After performing the indicated arithmetical computations, the crown thrusts and moments are obtained and from these the stresses are computed. Any designer who would be entrusted with a similar job for a right arch should be able to do this. Considerable thought was expended in an attempt to shorten the formulas, but in the general case none of the terms can well be omitted.

The practical value of this paper to the engineer lies in the demonstration that the skew arch can be analyzed for vertical loads as easily as a right arch, but under certain conditions only; the writer has indicated the approximations in this method and issued a warning against using the method if the approximations are too great, or if the loads do not have the nice adjustment indicated.

The loads producing the distress in an arch ring, then, are those that deviate most from the system which produces a thrust line applied at the junction of the neutral surface and the plane midway between the spandrel walls. The principal loads that need be considered are: (*a*) thrust due to temperature; (*b*) horizontal thrust from earth fill; (*c*) vertical loads eccentrically placed with respect to the crown, but midway between spandrel walls; and (*d*) vertical loads lying outside this mid-plane. In a given design, no doubt, some of these loads need not be analyzed, but in general they should all be considered. In the cases that have come to the writer's attention, the Loads (*b*) have been the cause of failure. Professor Beyer has given in his discussion§ a

* Asst. Prof., Civ. Eng., Univ. of Washington, Seattle, Wash.

† Received by the Secretary, April 2, 1923.

‡ See p. 682.

§ See p. 681.

qualitative idea of the effect of this type of loading, and indicates the complicated nature of the action. It is the writer's opinion that an engineer who designs a skew arch which is to be subjected to the torque due to horizontal loads should perform all the computations indicated in the paper.

It may be of interest to note that the thrust due to temperature acts nearer the acute than the obtuse corner (viewing the parallelogram of the arch in plan), in the arches examined in the paper, that is, for a rise of temperature the effect is opposite that indicated for loads by the late Professor Kernot.* The computations for this thrust are not lengthy.

If the concentrated vertical load is large and the fill moderate, Loads (*c*) and (*d*) may be quite serious. Load (*d*) presents a problem not peculiar to the skew arch, as it also arises in the case of the right arch. These loads could easily occur in highway work under heavy truck traffic.

After analyzing one or two skew arches, an engineer should be able to foresee the effect of loads as he now does in the case of the right arch, except that to visualize in three dimensions is more difficult.

The writer is pleased to make reply to the following points brought out by those who have discussed his paper:

(1) Dividing the arch ring by construction joints, as suggested by Mr. Hayden and Mr. Godfrey.†

(2) Dodging the problem by "camouflage", as advocated by Mr. Godfrey.

(3) The experiments of Professor Kernot.

(4) Dr. Fischer's paper.

(5) Dr. Marcus' solution.

(6) Mr. Harder's solution.

(7) The statement that the stresses take "the nearest route to the abutments", by Professor Kernot and Mr. Godfrey.

(8) The opinion of Mr. Godfrey that the type of arch is inherently faulty.

(9) Remarks about the theory being difficult and abstruse by Professor Fuller§ and Mr. Peck.‖

(10) A request for a method of obtaining unit stresses, by Professor Fuller and Mr. Harder.

(11) An outline of the approximations by Professor Beyer.

(12) "Horse sense" solutions suggested by Mr. Harder.

(13) Criticism of the position of the axes by Mr. Harder.

(14) Conclusions from Mr. Harder's Fig. 12.¶

(15) Remarks on the formula for torsional rigidity on page 140** by Professor Beyer and Mr. Harder.

(1) The writer fails to see anything gained in dividing the arch ring by joints in vertical planes parallel to the spandrel walls. For those loads that

* *Engineering News*, June 11, 1903.

† *Proceedings*, Am. Soc. C. E., April, 1924, p. 559.

‡ See p. 685.

§ *Proceedings*, Am. Soc. C. E., April, 1924, p. 559.

‖ See p. 698.

¶ See p. 688.

** *Proceedings*, Am. Soc. C. E., February, 1924.

cause no stress along these planes, such as the dead vertical loads, no good is accomplished and probably little or no harm. However, the arch ring is materially weakened in ability to carry the horizontal earth pressure or the reactions from spandrel walls. This method of construction was one of the causes of the failure of the skew arch in Tacoma, Wash.

(2) "Camouflage" has nothing to do with the analysis of stress in the arch ring, nor has the subject of "basket handle" arches.

(3) Professor Kernot's experiments, as described,* are of considerable interest; his deductions therefrom are still more so. It is surprising that these deductions were made. It is not clear to the writer how he performed the electrical experiment nor what character or position of loads was indicated by the result. The experiment with the rubber sling can be readily performed by hanging a strip of paper from two wires in such a manner as to represent an inverted skew arch. If this sling is loaded, the distribution of stress along the wire is plainly shown to be uniform, unless one of the wires is moved lengthwise slightly, in which case one can get any result desired. The paper has a tendency to slip along the wire, and if this is allowed the results obtained by Professor Kernot are developed.

The convincing experiment of the flat slab on which Professor Kernot based most of his theory may be duplicated in the drafting-room. Support a T-square blade on two triangular scales placed at the third points on the blade. The scales should be parallel and at an oblique angle to the blade, supporting, in effect, a skew slab. Load it in the center until it deflects appreciably and note, as did Professor Kernot, that the blade bears only on the obtuse corners of the slab. Now, bring the ends of the blade down until it is level at the place where it crosses the scales, thus representing a slab deflected with no rotation at the abutments. This is the condition imposed on arches. Now, note the stress distribution along the scale.

This experiment is developed mathematically in the writer's paper. Professor Beyer mentions it in his discussion with approval. The line of reasoning leading to the conclusions in Professor Kernot's paper is apparently convincing, but will not stand rigid investigation. The ring shortening due to direct thrust has only a small effect on the deformation and resulting stress distribution.

Regarding the recommendations of Professor Kernot, although it may have been good practice twenty-one years ago, this makeshift in skew arch design has no place in engineering to-day. If it is impossible to locate the stresses and estimate their magnitude, there is a possibility that more harm than good is being done by thus changing the ring. The stresses in statically indeterminate structures cannot be taken care of so easily.

(4) A reading of Dr. Fischer's original paper shows that he assumes the stresses to lie in a plane normal to the abutment. As this assumption is unwarranted, his theory is of little value. This was pointed out by Professor Beyer.

(5) The writer has been unable to obtain a copy of the paper by Dr. Marcus. As exemplified by Mr. Harder, it is evidently inapplicable to the majority of

* *Engineering News*, June 11, 1903.

skew arches, especially those with a spandrel fill, as the shaded triangles (Fig. 13*), which is assumed to be rigid, will form too large a percentage of the ring. It is gratifying to note that Mr. Harder has obtained the writer's equations from the general equations of Dr. Marcus. This was doubtless done after correcting for the inaccuracy introduced by the rigid triangles.

(6) Mr. Harder's solution is a distinct contribution to the subject, as he presents in a clear manner another method of attack, and, by an entirely different method of reasoning, arrives at the same results as the writer, for one system of loading. Doubtless, this same method can be expanded to include the other systems of loading to which a skew arch is subjected.

Both Professor Beyer and Mr. Harder bring out clearly that the writer has submitted a correct mathematical solution. After the formulas have been developed by Mr. Harder, the determination as to which is the most practical method resolves itself into a question of which can be understood and applied with the least labor. To compare these, then, requires the complete solution of a given example, including horizontal loads. Mr. Harder has used the idea of vectors and also the "work equation". For those who are familiar with this method of computation, the writer has given elsewhere—under Point (9)—a paragraph showing the ease with which his formulas may be derived.

A comparison of the results of the analysis of the arch of Fig. 17† is very interesting. That the methods of the writer and of Mr. Harder check, is not surprising, as the same assumptions and approximations were made by both. Within reasonable accuracy, $T_x = \epsilon\, T_z$ and $M_z = -\,\epsilon\, M_x$. This brings the crown reaction and the applied load, and, therefore, the abutment reaction, in the vertical plane half-way between the spandrel walls, a result that could have been anticipated.‡ As would be expected, Dr. Marcus' approximation caused considerable deviation from these results.

Comparing an abutment designed by Professor Kernot's method or by Dr. Marcus' method with one designed to take the reactions as computed by Mr. Harder or the writer, the first would be made heavy on one end; the second, heavy on the other; while the others would be designed with a uniform section.

(7) It is surprising that Mr. Godfrey publishes the statement that the stresses follow the nearest route to the abutment without furnishing a proof or showing wherein the proof of the writer is in error.

(8) The fifth paragraph of Mr. Godfrey's discussion does not state the inherent fault that this type of arch possesses, unless it is that heretofore a method of stress analysis has not been available. This fact no doubt accounts for the failures referred to. The idea that all structural failures can be classified into a few types is one that if developed would be a boon to the Engineering Profession. A paper on the subject with its discussions would be valuable in the extreme.

(9) As stated previously, the writer had in mind the development of the theory in a manner readily followed. Whether it was wiser to frighten the reader with a seemingly endless series of equations or to discourage him by

* See p. 688.

† See p. 689.

‡ *Proceedings*, Am. Soc. C. E., February, 1924, pp. 142-143.

using a method more difficult to understand was a dilemma. From Mr. Peck's discussion, the writer would infer that he had yet a long way to go to reach the ideal. It is hoped that familiarity of the profession with this solution will soon reduce it to a more workable form. Much algebra could have been avoided by the following method, although the resulting equations would have been the same.

Assume the ring to be rigidly fixed at the abutments and the crown reactions to be as given in Fig. 1.* The deformations in any section of the ring due to these reactions are as given in Equations (27),† which equations should be increased to include the loading terms obtained from Fig. 5‡ or Fig. 6.‡ Considering the arch ring rigidly fixed at the left abutment and acting as a cantilever, write a set of equations for the forces and moments in a section of the ring due to a unit force at the right abutment, acting along each axis in turn. By doing the same for a unit moment about each axis, a set of six equations is obtained. Multiply the terms of one set of equations by the corresponding terms of the other, in order to produce an equation of deflection of the right abutment relative to the left according to Castigliano's Law. Equations 1(*a*) to 1(*f*)§ are produced after cancelling out a large number of terms which become zero in a symmetrical arch or for other causes. This method is much shorter than the one given by the writer.

(10) Regarding the computation of unit stresses: Equations (26)‖ give the forces and moments due to the crown reactions acting on a plane perpendicular to the neutral surface of the arch and parallel to the Z-axis, that is, the normal plane. The forces and moments due to the loading when the section acts as a cantilever can readily be written from Fig. 5, Fig. 6, or a similar diagram. Given the forces acting on this plane, it is necessary only to know the law of distribution in order to compute the unit stresses. If the usual assumption is made—that a plane before bending remains a plane after bending—the distribution of stresses due to m_z and m_u are the same as in the simple beam theory. A slight error is introduced here, as the section of the arch ring cut by the plane is not, in general, a rectangle and the axes, Z and U, are not conjugate for the section cut. The shear, t_u, follows the parabolic distribution, as in the beam. The distribution of t_z is a trifle more complicated. It is to be noted that a force acting tangent to the neutral surface at the point, P, and lying in the plane midway between spandrel walls, produces no change in moment or torque across the normal plane through P. Therefore, the thrust and the shear due to such a force are distributed uniformly. The amount of this shear can be readily calculated, it being the Z-component of the tangent force mentioned previously. The value of this force should be taken so that its component perpendicular to the normal plane is equal to the calculated thrust on this plane, t_v. The remaining portion of t_z produces moment and follows the parabolic distribution. The law of distribution of the shear stresses due to m_v is very complicated, but for the maximum shear, which occurs at

* *Proceedings*, Am. Soc. C. E., February, 1924, p. 135.
† *Loc. cit.*, pp. 159–160.
‡ *Loc. cit.*, p. 144.
§ *Loc. cit.*, pp. 139–140.
‖ *Loc. cit.*, p. 159.

the mid-plane and on the extrados and intrados, the reference given on page 140* may be consulted, also numerous works on elasticity and resistance of materials of engineering, such as Johnson, Merriman, Wood, Slocum, and Burr. In this connection, it is to be noted that in the design of an arch all the unit stresses will be small, except those due to t_v, to m_z, to the part of t_z that is uniformly distributed, and to m_v. There is no question about the law of distribution of any of these except the last.

(11) The inaccuracies of the assumptions made by the writer are discussed by Professor Beyer. If the designer wishes to be more accurate in the computation of deformation due to shear by assuming a parabolic rather than a uniform distribution, he should merely increase the deformation 20% as shown in standard works on stress analysis. As all terms involving shear will usually be dropped, this is not an important matter unless it is a factor in laboratory tests on rings of a peculiar shape. The writer concurs with Professor Beyer that there is a question whether a torsion formula devised for a flat slab should be applied to a curved surface. However, as the torque has been taken block by block and as the curvature contained in any one block is slight, the writer feels, as does Professor Beyer, that he is justified. The fixed ends are a similar case. In arch design, it is well known that variations in thickness, etc., at the abutment do not have as great an effect as an error of the same amount at the crown. The arch ring tapers gradually toward the crown, and it is the thin and comparatively limber part of the ring that has the greatest effect, so the assumption that the ends are fixed should affect the formula but slightly.

(12) Unfortunately, Mr. Harder has not retained the faith in his method that it deserves, for he states in his conclusion that "the rules laid down by the late Professor Kernot contain more horse sense than all the mathematics that has come to his attention". Although the writer is a firm believer in good judgment in engineering, it is his opinion that stress analysis should be placed on a scientific basis and the dictates of unguided judgment followed only when no other method is available, and, even then, with great caution. This example of the skew arch, where scientific investigation shows that the "horse sense" method is materially on the side of danger, is a case in point.

(13) The writer has chosen the *X, Y,* and *Z*-axes as axes of reference, also the *u, v,* and *z*-axes. As co-ordinates of a point referred to one system are related to those of the other by the identities, Equations (3) and (4),† no approximation or error is introduced. The system is mathematically exact. Another system may be more convenient for another method of solution, but this system is a correct one as used.

(14) Mr. Harder's Fig. 12 and his accompanying discussion contain the very obvious error that all the forces are not shown. If T_z is shown as well as T_x and the resultant taken, the diagram will be materially changed. It will then be seen that the "anomaly which is obviously wrong" does not exist any more than it does in a column under compression when cut by an (imaginary) diagonal plane. The two examples are parallel.

* *Proceedings*, Am. Soc. C. E., February, 1924.
† *Loc. cit.*, p. 142.

(15) The writer is keenly disappointed that the discussion did not bring out some advance work or theories on torsion of flat slabs, as he feels that the accuracy of this formula (Equation (2)*) will control the accuracy of the determination of unit stresses due to eccentric loads. It is not a serious matter, however, as it has been shown that these stresses are only a small fraction of the whole and the engineer can adjust his formula so as to be on the safe side, without materially affecting his design.

An approximate formula can be used, therefore, and in writing this paper, it was hoped that some engineer who had made a study of the question would bring out some practical ideas. This formula was developed by Saint-Venant, a mathematician, and applies to all rectangles, from the square to one the side ratio of which is infinite. If the engineer fears to use the approximate formulas given by Mr. Harder or the writer, he can check these against the original formulas and correct them. Saint-Venant's formulas are too unwieldly for practical use; the writer does not feel that the labor involved is justified, except in special cases. Incidentally, Mr. Harder's assumption "that all planes remain planes after they are subjected to torsion", is in error and inconsistent with his assumption for the value of F. Mr. Harder's statement as to the time wasted in making calculations on the arch ring discussed, shows that he has missed the point of the paper, for if the writer's conclusions are true—and they have not been refuted—this arch is as correctly analyzed as the right arch of the same general dimensions can be, for all except a few of the loads, and the error involved in analyzing these loads is far less than that made in estimating them.

* *Proceedings*, Am. Soc. C. E., February, 1924, p. 140.

INCREASING THE CAPACITY OF EXISTING STREETS

Discussion*

By Messrs. Harvey W. Corbett, C. D. Hill and Jacob L. Crane, Jr., Harland Bartholomew, T. Kennard Thomson, W. W. Crosby, George S. Davison, and George T. Seabury.

Harvey W. Corbett,† Esq.—Mr. Tuttle has presented an interesting paper, but he does not go far enough. He blames traffic on the automobile. Everybody blames the automobile. The automobile is like the weather Mark Twain spoke about, "Everybody talks about it, but nobody does anything to improve it." The automobile is not the only cause to which traffic conditions are attributable.

It is a common statement that high buildings are erected in New York because there is no room for expansion anywhere except upward; the lower end of Manhattan appears to bear out this impression. One can readily understand that the traffic congestion is not wholly a question of automobiles or people; it also concerns the bulk of the building in relation to the street. The street is the artery which feeds the building with people, goods, food, etc. If the bulk of the building passes beyond a certain relation to the street, congestion is bound to result. Each building erected is five, ten, or even twenty times the bulk of the one it replaces, yet nothing is done with the street. That condition must be met; whether it is right or wrong, it is a condition. What can be done to increase the capacity of the existing streets? The city carries three easily divisible kinds of traffic: Foot, wheel, and rail. If the traffic problem is to be solved, logical divisions between those three types of traffic which do not belong together must be made.

For many years, grade crossings have been considered as a menace. It seems logical that rail, wheel, and foot traffic should be separated and placed in such relation to each other that they do not cross and conflict. As rail traffic is the heaviest in weight, it should go underground. It is already underground in the subways. The operation of surface-car lines is becoming less and less successful as a business proposition, and will be abandoned in time. Wheel traffic is gradually absorbing what formerly was carried by rail, because more people are riding in motors, and less on street cars and in subways. It is also absorbing foot traffic; there are more people riding than walking. Wheel traffic is the one great, growing form of traffic. Even freight which in past years was hauled in trains, is being carried not only into the city, but after it arrives there, in wheel vehicles. The solution of the difficulty is,

* This discussion (of the paper by Arthur S. Tuttle, M. Am. Soc. C. E., presented at the meeting of the City Planning Division on January 17, 1924, and published on p. 593 of this number of *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Archt. (Helmle & Corbett), New York, N. Y.

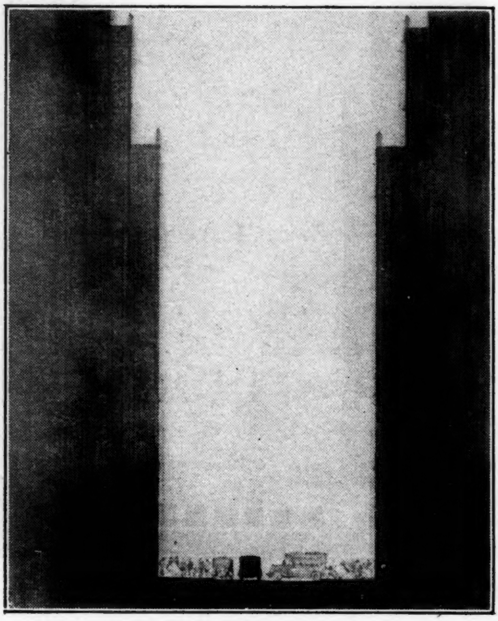


FIG. 6.—VIEW OF ORDINARY BUSINESS STREET AT PEAK LOAD.

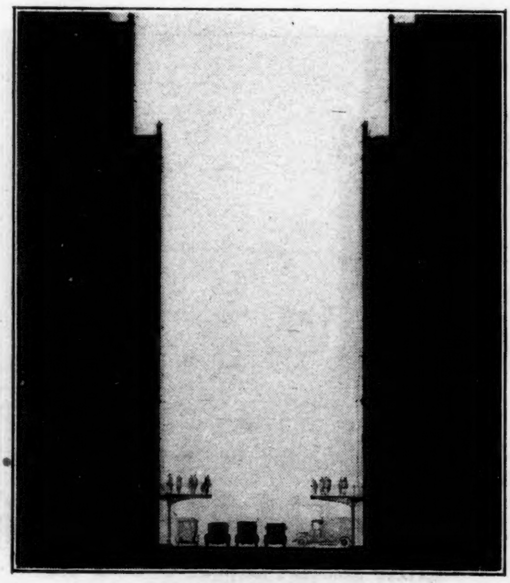


FIG. 7.—VIEW SHOWING PROPOSED ELEVATED SIDEWALKS.

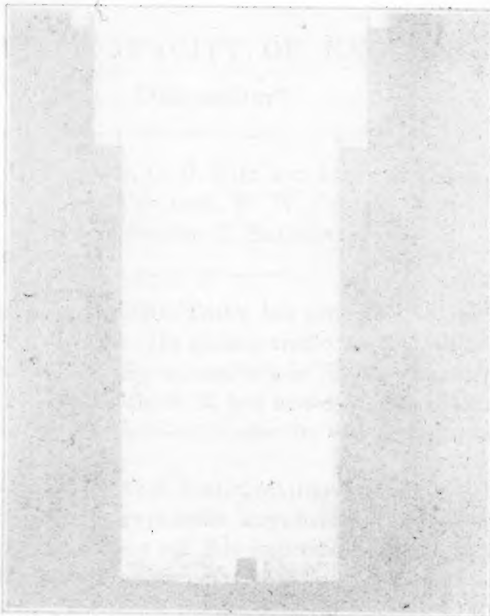


FIG. 6.—VIEW OF ORDINARY BUSINESS STREET AT PEAK LOAD.

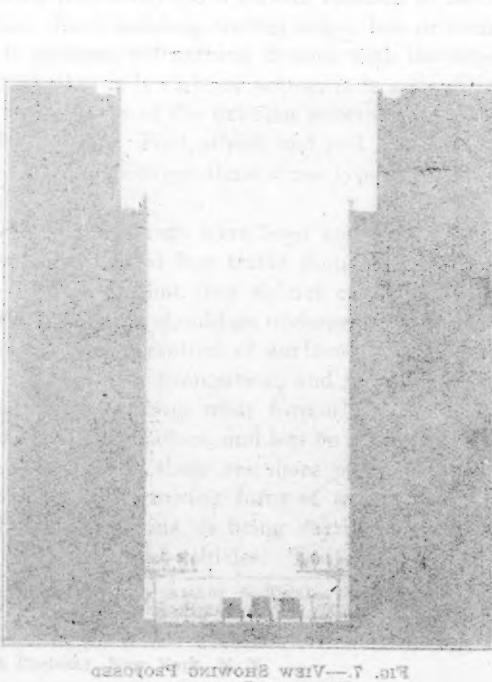


FIG. 7.—VIEW SHOWING PROPOSED ELEVATED SIDEWALKS.

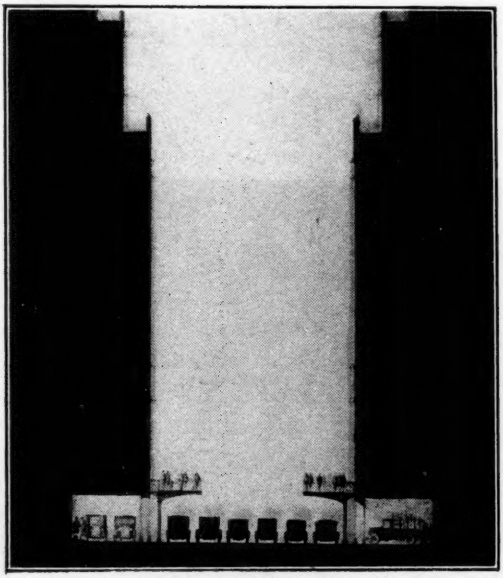


FIG. 8.—VIEW SHOWING PROVISION FOR PARKING SPACE UNDERNEATH BUILDINGS.

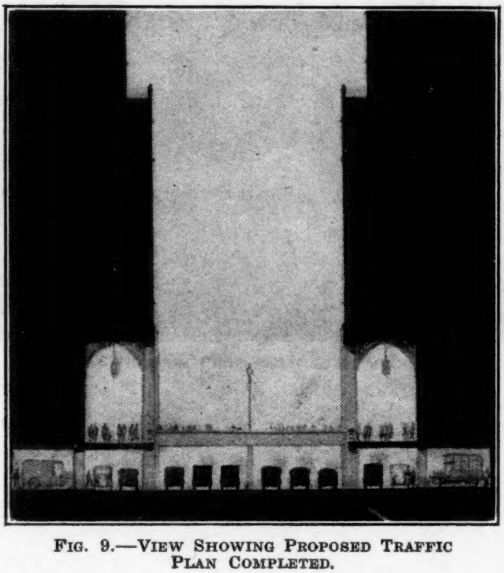


FIG. 9.—VIEW SHOWING PROPOSED TRAFFIC PLAN COMPLETED.

FIG. 10.—VIEW OF PROPOSED ARCADED CITY STREETS.

P

th
an
li

N
th
ro
st
ne
of
by

br
in
m
20

th

lev
rec
wi

an
pi
to
In
Au
rig
an
tra

Th
ges

fur
sol

Soc
str
eig
of
ity
tiv

therefore, simple: Use the present surface of the city for wheel traffic alone, and raise the pedestrian traffic one story above the wheel traffic, as it is the lightest and easiest to lift.

Fig. 6 shows the condition of an ordinary business street at peak load. Note the packages, bales, etc., with pedestrians trying to wiggle their way through, and a truck backed up to the curb cutting off three-fourths of the road. This can be seen on any of the cross streets at any time. A one-way street accommodates one line of traffic, as shown by the black car. It is necessary for trucks and vans to be parked, because that is part of the business of transportation of goods. It is an essential condition and cannot be changed by regulation or law.

Fig. 7 illustrates the first step proposed, that of placing elevated sidewalks, brought up from the grade at the avenues, thus allowing the trucks to park in the space formerly used for sidewalks. Such an arrangement would permit the movement of three lines of cars, which would increase the capacity 200 per cent.

Fig. 8 shows the third step, that of providing parking space underneath the building, thus increasing the traffic to six moving lines.

Fig. 9 shows the completed plan, the foot traffic remaining at the upper level, with the wheel traffic at the lower level. Building regulations should require all future buildings to be designed to provide for arcaded sidewalks within the building level.

Fig. 10 shows a city in its final aspect. The people all go to the upper level and walk through the city undisturbed by traffic conditions. The whole shopping zone would be available for them. The smaller parks would be raised to this same level, the space underneath being used as public parking space. In this arcaded city, the cross streets would pass under the avenues as shown. Automobiles crossing the city would take the lower level. By using only right-hand turns and under-cutting the avenues, it would be possible to reach any point of the city without being stopped, except by the slowing up due to traffic moving in the same direction.

It is difficult to approximate what the increase of traffic capacity would be. The speaker is satisfied that 2 000% increase of traffic over the present congested condition, is no exaggeration for such an improvement.

In New York and in all other cities where the traffic problem is acute, some fundamental principle should be established by which every proposed traffic solution should be tested.

C. D. Hill,* M. Am. Soc. C. E., and Jacob L. Crane, Jr.,† Assoc. M. Am. Soc. C. E.—It is desired to present herein information concerning two-level streets in Chicago, Ill. One two-level street, five blocks long, is in use; another, eight blocks long, is definitely projected; and, in addition, there are a number of unofficial proposals for the congested district. By checking the total capacity of one-level and two-level streets against their respective costs, the relative traffic-carrying efficiencies become apparent.

* Chicago, Ill.

† Municipal Development Engr., Chicago, Ill.

The principal elements of cost in these improvements, one-level as compared to two-level, are as follows:

	Land.	Construction.	Total.
Michigan Avenue, Two-Level......	$5 000 000	$10 000 000	$15 000 000
Michigan Avenue, One-Level......	5 000 000	4 000 000	9 000 000
South Water Street, Two-Level....	11 000 000	11 000 000	22 000 000
South Water Street, One-Level....	7 000 000	4 000 000	11 000 000

The nature of the foundation soil in Chicago necessitates elaborate and expensive systems of footings, extending as deep as 80 ft. below the grade of the lower level, to support the columns under the upper level. This accounts for a large part of the extra construction cost in the existing and proposed two-level streets.

Michigan Avenue Two-Level Street.—The completed two-level street, known as the Michigan Avenue Improvement, or "Boulevard Link", was constructed to connect the upper end of Michigan Avenue south of the Chicago River directly through to the Lake Shore Drive, which, in turn, is a through boulevard to the northern end of the city and the north shore suburbs. Formerly, the heavy traffic flowing from Michigan Avenue to Lake Shore Drive was compelled to take a narrow and devious by-pass over the Rush Street Bridge. The new connection (Figs. 11 and 12)* eliminates the by-pass and furnishes a wide, direct roadway, increasing many-fold the carrying capacity of this most important artery to and from the business center. Practically all the passenger traffic passes along the upper level, making it one of the greatest traffic carriers in the world. It accommodates more than 35 000 vehicles per day; within ten years, it will probably be carrying twice that number.

The carrying capacity of this upper level, however, would be much less if it were not for the lower-level arrangement, which removes the obstruction of a heavy traffic of teams and trucks along the line of Michigan Avenue and particularly across it. The largest single element in the cross-boulevard commercial traffic is that flowing to and from the freight yards of the Illinois Central and New York Central Railroads. From 4 000 to 5 000 tons of L. C. L. freight are hauled in and out of these yards each day, and the resulting traffic amounts to 5 000 or 6 000 teams, trucks, and automobiles. This traffic does not continue through the evening period of peak density for the upper-level traffic; still, if it passed across the upper level, it would reduce by one-third the capacity of that roadway. The lower-level traffic at the remaining cross-streets—Grand Avenue, Illinois, Austin, and North Water—totals another 5 000 vehicles, most of which are slow-moving. There is also a street-car line on Grand Avenue, and a railroad switching line on North Water Street. If this additional lower-level traffic was allowed to flow across and along the upper level, it would still further reduce its capacity.

The main purpose, therefore, of this two-level Michigan Avenue Improvement is the separation of the fast passenger vehicle traffic on the Boulevard from the slow commercial traffic which now moves along and across the lower

* Furnished by the courtesy of the Chicago Plan Commission.

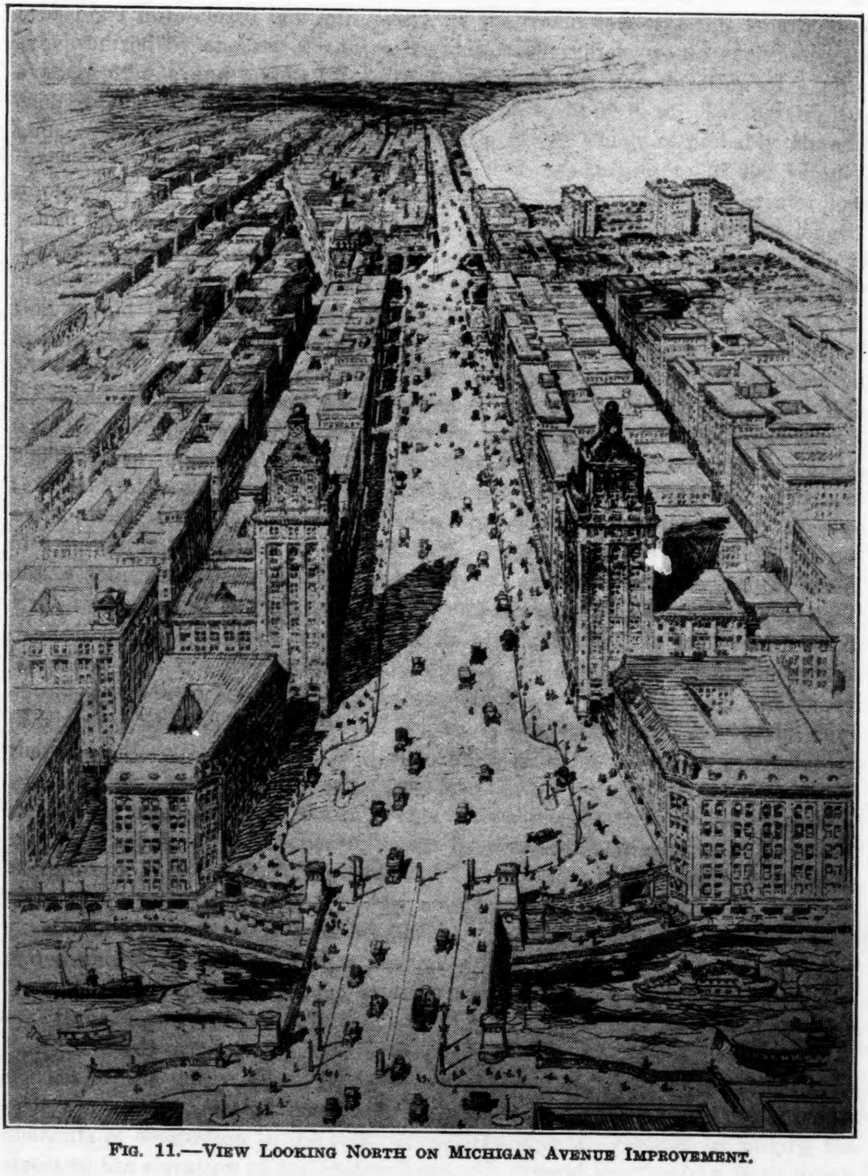


Fig. 11.—View Looking North on Michigan Avenue Improvement.

level. In order to measure the money value of the time saved to the upper-level traffic by this separation, the following calculation has been made: To allow the lower-level traffic to flow across Michigan Avenue at the upper level, would make it necessary to interrupt the upper-level flow on the schedule of probably 1 min. closed to 2 min. open. If the traffic flow over this length were synchronized by one set of control signals, each vehicle would experience an average of one and one-half interruptions of an average duration of ½ min. each, during the working day, from 7:00 A. M. to 5:00 P. M. Taking this 10-hour upper-level flow at 36 000 vehicles (a figure which is probably already exceeded), there is an aggregate loss of time amounting to 27 000 vehicle-minutes or 450 vehicle-hours per day. At the same time, a still greater loss would be caused by the slowing down of all machines passing over this length. The average speed now attained with the upper level free from cross traffic is about 25 miles per hour, compared with an average of about 15 miles per hour that would be possible if this were a one-level improvement. This factor accounts for an average saving of 1 min., and gives an aggregate saving of 36 000 vehicle-minutes, or of 600 vehicle-hours, per day. The total time saved is, therefore, not less than 1 000 vehicle-hours per day. By evaluating this time at $2.50 per hour for vehicle, driver, and passenger, extending it over a 300-day year, and capitalizing this amount at 7% to cover interest, depreciation, and maintenance, a total of more than $10 000 000 is obtained. Furthermore, there is the saving to lower-level traffic, which amounts to an average of 1 min. for two-thirds of all the traffic. Using a traffic flow of 15 000 per day, a total of 10 000 vehicle-minutes, or 167 vehicle-hours, per day is saved. Evaluating this at $3 per vehicle-hour, extending it over a 300-day year, and capitalizing at 7%, gives another saving of $2 000 000. It might be argued, therefore, that the city would be justified in spending at least $12 000 000 to obtain a two-level instead of a one-level street over this distance. As shown previously, however, the two-level improvement costs only $6 000 000 more than a one-level street would have cost. This extra cost is all in construction, which is two and one-half times as much for a two-level as for a one-level street. In contrast to the South Water Street project, however, no extra land was required in this instance to make the two levels possible. It is found, therefore, that, for this "Boulevard Link" Improvement, the two-level arrangement is entirely justified..

South Water Street.—This projected two-level street (Figs. 13 and 14) is called the South Water Street Improvement. Its objective is many-sided. Even the one-level improvement originally contemplated would accomplish several important purposes: First, it would remove from South Water Street a badly located, overcrowded, and inefficient Commission Market, the principal market of the entire city. In addition to the street obstruction due to the operation of the Market, the traffic to it and from it is one of the major elements of congestion in the other down-town streets. Second, by raising the street to the elevation of the bridge floors, the present steep ramps from South Water Street to the bridges would be eliminated. Third, this improvement would provide a by-pass around the painfully congested "Loop" streets for traffic going west or southwest, and *vice versa;* also, it would serve to dis-

tribute traffic through the "Loop" as from a header or feeder. Fourth, it would offer opportunity for the improvement of dilapidated river-front property and for a monumental boulevard treatment of the water-front itself.

Such a one-level street would have sufficient capacity for all the combined traffic that both levels of the proposed project are likely to receive. In that case, however, as may be seen from Fig. 13,* the South Water Street Improvement could not connect with the upper level of Michigan Avenue and, at the same time, carry away the heavy commercial traffic flowing from the lower level of Michigan Avenue. It is extremely important to have this lower-level traffic carried on South Water Street so as to avoid the "Loop" streets.

However, the lower level of the proposed South Water Street Improvement will serve for more than an outlet to the Michigan Avenue lower level. Although it will carry this heavy traffic without interruption at cross-streets over a separate right of way around the congested business center, it will also eliminate the otherwise serious factor of delay caused by crossing the line of each of the upper-level streets passing out of the "Loop". This is particularly important where it crosses La Salle Street, now proposed as a major artery, similar to Michigan Avenue. Finally, the lower level has some value for commercial wharf purposes along the river.

If this lower level along South Water Street was not required to co-ordinate with the Michigan Avenue improvement,† would the two-level structure be justified? It is officially estimated that the two-level street will cost about $22 000 000, whereas the same improvement carried out on one level only is estimated to cost about $11 000 000, or 50% as much. In contrast to the situation at Michigan Avenue, two-thirds of the excess cost is necessary in this case for extra land.

This extra cost of $11 000 000 will separate perhaps 6 000 commercial vehicles per day on the lower level from, perhaps, 10 000, or even 20 000, passenger vehicles per day on the upper level.

It is difficult to estimate accurately the saving in time which will accrue to vehicles by virtue of the two-level separation, but it can be approximated. About two-thirds of the lower-level traffic will continue over the full length of the improvement. The delay, therefore, to this traffic would not exceed, for a one-and-one schedule at cross-streets, an average of ½ min. each for one-half the traffic at each of seven cross-streets, or a total of 175 vehicle-hours per day. The reduction in average speed from 20 to 10 miles per hour for two-thirds of the traffic would account for an additional 150 vehicle-hours per day. Applying these figures to a 300-day year, and capitalizing at 7%, the sum of about $4 000 000 is obtained. The upper-level cross-street traffic, totaling, perhaps, 30 000 vehicles per day on all the seven streets, would be saved, on an average, not more than ½ min. each for one-third of the vehicles. This produces about 85 vehicle-hours per day and capitalizes to a saving of another $1 000 000. The saving to traffic effected by the South Water Street

* A drawing of the Chicago Plan Commission.

† The plans for the "Boulevard Link" were completed in 1913, before the plans for South Water Street were formulated.

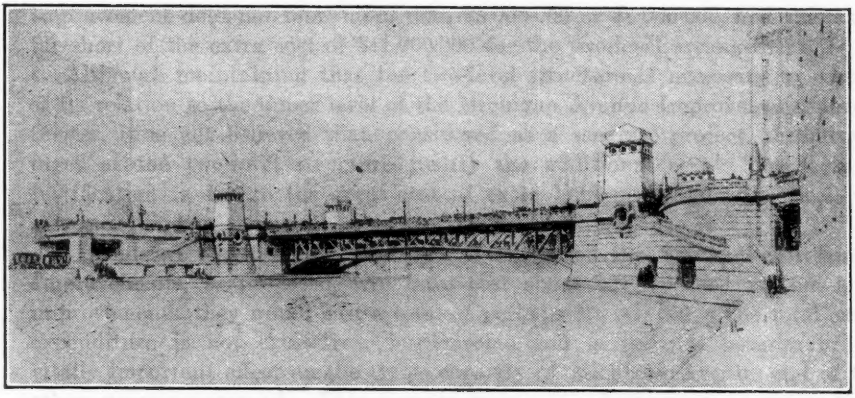


FIG. 12.—VIEW OF TWO-LEVEL MICHIGAN AVENUE BRIDGE.

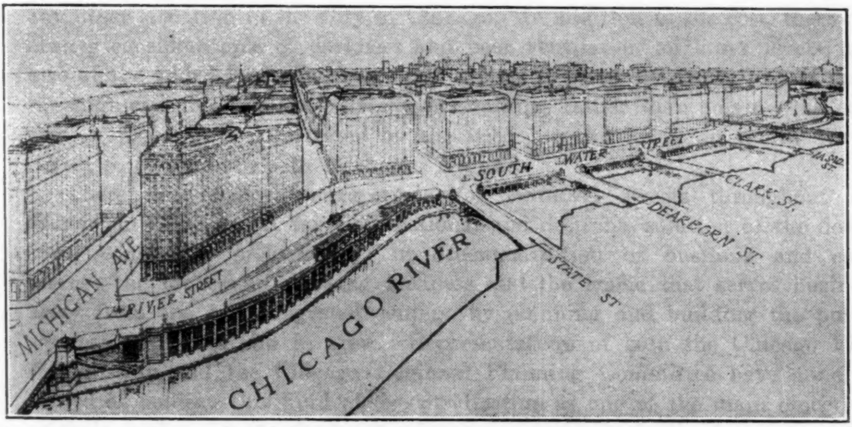


FIG. 13.—THE SOUTH WATER STREET IMPROVEMENT (PROPOSED), EXTENDING WEST FROM MICHIGAN AVENUE.

FIG. 14.—THE SOUTH WATER STREET IMPROVEMENT (PROPOSED) IN CROSS-SECTION.

Improvement does not total more than \$5 000 000 or \$6 000 000, and this falls far short of the extra cost of \$11 000 000 for the two-level arrangement.

Although maintaining that the two-level structure is necessary by virtue of its relation to the upper level of the Michigan Avenue Improvement, nevertheless, it is not believed that, considered as a separate project, the advantages of the two-level structure justify the additional cost. The lack of justification is due to the great cost of extra land needed for the two-level structure, and to the small volume of traffic which can use it.

Considered together, the Michigan Avenue and the South Water Street Improvements, as proposed, will have cost about \$37 000 000. As one-level improvements, they would cost a total of perhaps \$20 000 000. The total extra expenditure is not excessively burdensome and is justified because of its vitally important effect on the traffic capacity of Michigan Avenue and of the other streets concerned. This is true, however, only because of the critical importance of the arteries involved; probably it would not be justified for any other situation in the City of Chicago. In addition to the cost, there are always considerations of darkness and poor ventilation on lower levels, and also of the added load placed on other streets to carry the traffic to and from the high-capacity two-level streets, pyramiding rather than solving the congestion problem, as illustrated by the need for two levels on South Water Street to co-ordinate with Michigan Avenue.

There are frequent suggestions for two-level streets throughout the "Loop" in Chicago. A far more rational and desirable solution of the downtown congestion problem lies in decentralization of business and commerce—the process of drawing business and the traffic that serves business away from the over-congested centers, by planning and building the public services with this aim in view. Representatives of both the Chicago Plan Commission and the Chicago Regional Planning Committee have gone on record as holding this kind of decentralization as one of the main objectives of their work—to make the two-level street unnecessary in the future.

HARLAND BARTHOLOMEW,* M. AM. SOC. C. E.—Mr. Corbett's ingenious plans represent as interesting a solution of the problem of separation of various kinds of street traffic as could probably be devised. They are better by far than any actual work that would be done and illustrate how difficult and complex would be the problem of private property development. Their success is almost entirely dependent on co-operation and simultaneous improvement of a large number of adjoining properties—an extraordinarily difficult thing to accomplish.

Assuming that some city might find itself fortunate enough to accomplish and finance such a separation of street traffic, what problem would have been solved? More traffic would be accommodated, but it would be just a question of time before one, if not all, of the several street levels would be congested—and then what? More levels to accommodate more traffic? Where would be the end? This plan is certainly not a solution of the traffic problem; that must be found in a different method of approach.

---

* Engr., City Plan Comm., St. Louis, Mo.

No city has sufficient street space in its business districts to accommodate all its traffic. The volume of use is increasing, making the percentage of traffic that can be accommodated in the streets of business districts increasingly less. Even though a saturation point of motor-car ownership might be assumed, it would be financially impossible to provide for unlimited accommodation of all kinds of traffic.

The first step toward a solution of this problem is the reduction or elimination of unnecessary traffic movements. Studies recently made in several cities indicated that from 20 to 40% of the vehicle movement was commercial in character. The remainder consisted of individual pleasure cars, a small proportion of which conceivably might be engaged in strictly commercial use. Certainly, a large percentage of present street traffic bears little relation to the business development of the community. Such traffic should either be eliminated entirely from congested areas or at least be confined to a limited number of streets.

The second step toward a solution of this problem is to correct existing street widths and arrangement so as to permit the greatest possible freedom and circulation of traffic movement without inviting prohibitive expense.

The third and last step will be the decentralization of congested areas either by enlarging the area of central nuclei or the establishment of outlying sub-centers, or both.

There can be no complete solution of this problem without the limitation of the size and growth of cities. To invite the tremendous expense implied by Mr. Corbett's plans is to enter on an endless pyramiding of expense. The idea of constantly increasing the height of buildings and attempting to provide accommodation for an ever-increasing traffic at their base is making progress backwards and can lead only to economic strangulation.

The two instances of separated street grades in Chicago cited by Messrs. Hill and Crane are excellent illustrations of the economic impossibility of the universal application of Mr. Corbett's plans. There are not ten cities in the United States to-day that can afford rapid transit subways, for instance, and yet hundreds of cities have street traffic congestion problems that are becoming more and more acute, and which require immediate relief.

The fact that separated street grades will solve no traffic problem permanently is justification for the assertion that proponents of these should not look to public revenues for their execution. Before inviting huge public expenditures for work of this character, engineers should first ask how far a city's credit might be taxed for the purpose of securing increase in street capacity. When the huge demands already made on municipal revenues and credit are considered, it becomes increasingly clear that any city's financial ability to increase street space is extremely limited.

To repeat—the solution of the problem of street traffic congestion lies not in doing those things that invite greater concentration, but in doing those things that will lessen or distribute traffic, such as, (1) reducing unnecessary movements; (2) correction of defective means of circulation; and (3) enlargement of area, reduction of height, and decentralization of business districts.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—The Society is greatly indebted to Mr. Corbett for his valuable and remarkably well presented discussion. For more than thirteen years, the speaker has been working on a problem which is much simpler, inasmuch as Mr. Corbett's task is hampered by existing conditions, whereas that of the speaker is untrammelled by such obstacles. There is an opportunity to create an idealistic city 9 sq. miles in area, in the heart of New York, without destroying any existing property, simply by extending Manhattan 6 miles down the Bay.

Mr. Corbett's excellent suggestion that cross-streets pass above longitudinal streets would certainly be a great relief to the present congestion.

W. W. CROSBY,† M. AM. SOC. C. E.—The ideas suggested have been extremely interesting.

In Los Angeles where the speaker has been frequently, traffic conditions have improved, but are not yet such as to set an example for New York. The excellent suggestion made in regard to a platoon system of regulating traffic, has some defects in practice, as pointed out by Mr. Bartholomew, and those defects are decidedly in evidence in Los Angeles at present.

It must be acknowledged that, particularly in America, rules and regulations for pedestrians are difficult to enforce satisfactorily. Automatic or subconcious control works better, provided the automatic regulator is not too delicate or too likely to get "jammed." The great trouble in Los Angeles is the habit of pedestrians to ignore signals, and the tendency of other traffic to defy them.

The theory of separating foot and vehicular traffic appeals strongly as a matter of simple automatic control. It may be of interest to note that one of the oldest cities in England—the City of Chester—has double-decked arcaded ways for foot passengers, which would illustrate Mr. Tuttle's ideas. There, however, both stories are still used for foot passengers and the automobile has not yet appropriated the lower level for parking spaces.

The arcaded streets in Europe and elsewhere are well known and have succeeded in greatly increasing the vehicular capacity of the roadways.

In New York, an original through street might be compared to a feed-wire to which an excessive number of outlets have since been added. The question now becomes one of keeping down the additions or of enlarging the feed.

It may be remembered that in 1884 there was a proposal to build a subway under Fifth Avenue which was thus to be made a double-deck street and an important "feed". The problem in New York has been attacked and partly solved, but it seems that some repression of the demands on the streets will have to be enforced or else some suggestion such as that of Mr. Corbett will have to be adopted.

GEORGE S. DAVISON,‡ M. AM. SOC. C. E.—The word "repression" has been used in this discussion. The speaker thinks there has been, and, in the future, there will be, a great deal done in the way of repression that will help the con-

* Cons. Engr., New York, N. Y.
† Cons. Engr., Baltimore, Md.
‡ Pres., Gulf Refining Co., Pittsburgh, Pa.

dition of congestion in large cities. Other discussers have evidently in mind that the pleasure car business will grow, and, consequently, its traffic will continue to increase at the present rate. The speaker does not share that idea; he thinks there will soon be a slackening of this business as it affects congestion, and that finally the point of saturation will be reached. It does not seem possible that the automobile industry can keep on making cars at the present rate, to be absorbed by new customers in the United States. The manufacturers will work largely on substitutions for worn-out equipments, which naturally will not increase congestion. The demand for cars by people who have not previously owned them, must diminish materially; otherwise, the time would come when every adult would own and operate a car, which condition is not conceivable. So, generally speaking, the problem of providing for the future automobile traffic is not as serious as stated.

The problem in New York is different from that of Pittsburgh, Pa., where the speaker is a member of a committee which has recently been appointed to study the problem of street congestion. As it would appear at present, Pittsburgh cannot afford to have rapid transit subways; the best it could do would be to build subway loops for trolley cars in the congested part of the city. Building such loops, would relieve the congestion of the traffic in a small area known as the "Golden Triangle", but even then only by a few hundred street cars each twenty-four hours. This would not materially reduce the congestion, which in Pittsburgh, as elsewhere, arises through the use of the streets by thousands of pleasure automobiles and commercial trucks. Not the least of the difficulties arises from the parking of cars in double line along narrow business streets, thus retarding the flow of traffic on such sections of the roadways as are not occupied by these standing vehicles. The large department stores are located within this area, and the "lady of the house" adds her share of confusion by causing her car, which may represent only herself, or, at most, a family of three or four, to occupy 100 sq. ft. of space for an hour at a time. Meanwhile, the street car carrying 40 or 50 people, is occupying a moving area of not to exceed 400 sq. ft.

As far as the Committee has observed considerable automobile and truck traffic can be by-passed around the "Triangle" without any serious inconvenience, and those who now drive their cars into parking spaces within it can, with no additional discomfort to themselves, leave their cars at parking spaces at the edge.

There is an economic question involved in this whole matter, at least in Pittsburgh—the large sums of money which it may be necessary to expend for underground loops for street cars must have their interest payments carried for many years and eventually be amortized. Who shall be burdened with these payments? Surely not the street car rider, because he is now complaining, and with some reason, that he is paying too much at present. It is not his vehicle that is the primary cause of the congestion. If not he, then it must be the taxpayer. Arriving at that conclusion, it must be considered that comparatively few of the tax-paying class are involved in the reduction of congestion; these few are the automobile owners.

It would seem that there are two general plans on which to work. One would be to formulate better regulations for controlling the automobile and truck traffic in order that the occupation of the streets might be greatly reduced; and the other, planning street improvements for by-passing purposes, the automobile and truck owner carrying the cost. The speaker realizes that there must be new legislation enacted to cover the latter case, but believes there is fully as much reason for the vehicle owner to pay a license fee to the city wherein he resides, as that he should pay a fee to his State, from which his benefits are much more obscure than those that would inure to him in the maintenance of proper traffic conditions in his city.

George T. Seabury,* M. Am. Soc. C. E.—In their discussions, Messrs. Bartholomew and Davison have touched on a practical point. The idealistic plans discussed have been inspiring and some of the vision has been caught, but it may not be. Traffic problems are imminent and must be solved without delay and without considerable expenditure. Engineers may not dream visions; they must work out practical simple solutions and at once.

Like Mr. Davison, the speaker is one of a committee charged with devising a solution, applicable at once, to the congested existing streets of a busy city, namely, Providence, R. I. Trolley, vehicular, and pedestrian traffic is now using all the pavement not occupied by standing automobiles and all too frequently it finds the space insufficient.

It is evident that any practical solution must be based on the premise that most of the vehicles that wish to use the pavement area for purposes of transit must be allowed to do so. If the premise were that all vehicles that desired to move through the streets of the congested area should be allowed to do so, the most obvious solution would be the elimination of parking. It is not necessary, however, for all vehicles to use those streets and segregation may be made between streets primarily for trolleys and other streets without trolleys. It is practicable to re-route either the automobiles in large part, or the trolleys. In Providence, the latter has been accomplished recently, though with loud complaint—not entirely unjustified—from the trolley users. Admitting the possibility of re-routing even more, if necessary, the solution must depend on some other factor.

Providence is not so laid out that it suffers particularly from through traffic; at least, none of the streets that now need attention needs it for that reason except during the afternoon rush hours. Consequently, any attempted segregation of vehicles reduces rather to segregation of types than to segregation of separate streets. The mind at once jumps to the tentative decision that vehicles used for pleasure or, perhaps, even for convenience, should not be permitted in the center of the city. Such a conclusion is somewhat warranted by an analysis of the amount of space used by the several types of vehicles as compared with the justifiable use made of that space. It appears that the trolley car is used to move about 80% of the population that does business in the congested area and that the remaining 20% is moved by automobiles. It appears, moreover, that one trolley renders service to approximately 40 people,

* Pres. and Gen. Mgr., George T. Seabury, Inc., Providence, R. I.

whereas one automobile serves only about 2 people; in other words, a trolley is equivalent in carrying capacity to 20 automobiles. The street space needed by a moving trolley is something like 100 lin. ft., whereas the space on the same street needed for a moving automobile is from 20 to 25 ft. Therefore, the use of street space for the transportation of people by trolleys is more efficient than that by automobile in a ratio of at least 4 to 1. It is admitted that subways for trolleys would be advantageous, but subways are an impracticable solution on account of both time and money.

It has been shown that as between the trolley and the passenger automobile, the latter must yield; this conclusion is strengthened by the necessity of the use of the pavement by automobiles engaged in other than passenger traffic. It would inflict an almost impossible burden on the transaction of business to exclude commercial vehicles from the use of the streets not only for purposes of transit, but for loading and unloading. Here, again, the elimination of the parked passenger automobile seems the logical immediate solution. It is admitted that the commercial vehicle abuses its logical pre-eminence. The towel-supply truck, the ice cart, the spring-water delivery automobile all loiter along, and too frequenntly stand unused in front of a business house for long periods. In the main, however, the city is for business and the means of doing business must be given preference.

The much-to-be-encouraged business, however, does not confine itself to the use of any one type of automobile. Any general segregation based on the type of vehicle is, therefore, inapplicable to the great majority of streets. Thus, one is largely at sea as to a solution based on the foregoing line of reasoning. The premise in regard to the necessity of making the present streets adequate for the use of all vehicles remains.

Granting this premise and laying aside, for the moment, any opinions with regard to segregation of vehicles, etc., it seems there is a constructive method that should lead to a solution of the problem, as illustrated by the plan the Committee is following in its study in Providence.

Considering each street by itself, an attempt is being made to decide how many lanes of moving traffic are needed to permit the passage of all vehicles. Coincident with this, a decision is made as to whether or not it is possible to permit the standing of automobiles on that street. When the width of the existing pavement is such that the requisite number of moving lanes can be secured and, in addition, automobiles of either the commercial or passenger-carrying type may stand parked, well and good. On the other hand, if this is not possible, it is proposed to consider that the passenger type of vehicle may do no more than stop to take on or to let off passengers, nor the commercial vehicle do more than stop for the time necessary to load or unload with reasonable dispatch merchandise ready for loading or unloading.

By such a process of elimination it will undoubtedly develop that the permissible parking space will be curtailed. With ever more vehicles desiring to park and with less space in which to do so, the only equitable solution (unless adequate and convenient parking spaces in addition to the pavement are provided) seems to be a reduction in the time during which parking may be permitted. This solution, it is believed, will not of itself result in a completely

satisfactory use of the streets throughout the entire day. The number of moving lanes needed will vary with the hour. It is proposed, therefore, to base the number of moving lanes needed on the early afternoon traffic and to designate streets or zones in which, during the late afternoon rush hours, all vehicles shall be required to move. In this "all-rolling zone", both the parking of automobiles and the loading and delivery of merchandise will be eliminated except in emergency or by permit.

RESEARCHES ON THE STRUCTURAL DESIGN OF HIGHWAYS BY THE UNITED STATES BUREAU OF PUBLIC ROADS

Discussion*

By Prévost Hubbard, Affiliate, Am. Soc. C. E.

Prévost Hubbard,† Affiliate, Am. Soc. C. E.—The question has been raised as to the relation of smoothness of the concrete base to displacement of an asphalt top under traffic. The speaker has inspected the Arlington experiments (but not since any of the sections were taken up), and is interested in the progression of movement in the sections which shoved; whether the shoving apparently progressed from the top down or whether it was most pronounced at the point of contact with the base. He understands that metal plugs were buried at different points in the mixture for the purpose of determining the relative movements at different depths. It is thought this will be an important matter to determine, and may indicate whether or not the smoothness of the base has any effect upon shoving.

Another question is in connection with the impact test results. The basis of comparison of resistance of the different types of slabs appears to be the number of pounds of impact required to cause failure. The question arises as to whether this is the proper gauge to use for different classes of materials. The speaker believes it has been ascertained that a 5-ton truck running at a speed of 12 miles per hour, and falling from a height of 2 in., delivers an impact of about 28 700 lb. on a gravel road and about 36 800 lb. on a concrete road. This means that, the conditions of test being equal, the number of pounds of impact varied. Now, in comparing different types and designs of road slabs or road structures, on a basis of impact pounds delivered, will not this mean that the conditions causing the same number of impact pounds on the different types may be different—in other words, more severe in one case than another—and, if that is so, is it proper to make the basis of comparison entirely on impact pounds? Comparison should be made as regards the behavior of the different types subjected to the same conditions of test rather than as regards the intensity of blow causing failure.

Has any attempt been made to determine the effect of fatigue on the rigid types of construction under impact as has been done under static or suddenly applied loads? Is it not possible that fatigue may play a more important part in the resistance of rigid structures to impact than in their resistance to repeated static loading?

* The discussion (of the paper by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., presented at the meeting of the Highway Division on January 17, 1924, and published in April, 1924, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chemical Engr., The Asphalt Assoc., New York, N. Y.

SANITATION—ITS RELATION TO HEALTH AND LIFE
Discussion*

By Russell Suter, M. Am. Soc. C. E.

Russell Suter,† M. Am. Soc. C. E. (by letter).‡—Life is usually described as a struggle against environment. This description places undue emphasis on the adverse effects of environment, for, although life is terminated by the adverse, it is maintained by utilizing the favorable elements.

Health is the condition of being attuned to the environment. To maintain this condition, man particularly utilizes the services of two groups of specialists: the physician, whose duty it is to repair the damage caused by unfavorable environmental elements and to fit man to his environment; and the sanitary engineer, whose province is to change the environment to fit the man.

Public health-regulative bodies, the duty of which is to promote health and increase the span of life, must utilize the services of both these groups. Possibly the functions of the sanitary engineer in public health are, from their nature, of greater magnitude and scope than those of the physician. The latter primarily deals with individuals, or private matters, whereas the former handles matters of more general public import.

It is perhaps unfortunate that in the public mind anything relating to health has become closely connected with the physician, whose principal function is to restore health rather than to preserve it. Nevertheless, it seems to the writer that the term, "health", as applied to public departments, is a proper one. It must be recognized as fundamental that the activities of the sanitary engineer are at least as necessary to the administration of public health as those of the physician.

Unfortunately, the training, mental habits, and practices of the physician and the engineer are so at variance that they do not work well together and either is prone to assume an attitude of contempt toward the other. This condition shows itself in the discussion as to whether the head of a public health department should or should not be a physician. The writer does not consider that the requirement that a public health officer should be a doctor of medicine is wise, but he would not go so far as to state that the use of a physician in this position should be forbidden. It is an administrative office, the requirements for which should be administrative ability. Neither a physician nor a sanitary engineer can be a successful health administrator without proper

* This discussion (of the paper by George C. Whipple, M. Am. Soc. C. E., presented before the Sanitary Engineering Division on January 15, 1924, and published in April, 1924, *Proceedings*), is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Senior Asst. Engr., Conservation Comm., Albany, N. Y.

‡ Received by the Secretary, January 18, 1924.

preliminary training in that work. If he has suitable administrative capacity, he will know enough to assign technical questions to his technical advisers for settlement and to be guided by their opinions.

The writer would consider that administrative cleavage between public health and public sanitation would be most unfortunate, although in making that statement he appreciates that present conditions are far from ideal.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ADAM LEONARD BUSH, M. Am. Soc. C. E.*

DIED OCTOBER 21, 1923.

Adam Leonard Bush was born at Grange, Jefferson County, Pa., on February 22, 1869.

After learning the trade of carpentry, and teaching mathematics for some time, Mr. Bush entered the field of structural engineering by way of the templet shop. His mathematical ability and his experience in templet-making, soon attracted him to the structural drafting-rooms of the American Bridge Company, at Ambridge, Pa., and afterward into structural engineering.

A running narrative of Mr. Bush's experience, with dates and the positions that he held, could not do justice to his accomplishments. His career paralleled that of many structural engineers who have advanced from the intricacies of the skewed portal and sway-bracing of the drafting-room to the complications of the modern bridge and skyscraper.

Mr. Bush was Chief Engineer for Parkinson and Bergstrom, Architects, of Los Angeles, Calif., from 1909 until 1918, and designed the structural work for about twenty of the largest buildings in Los Angeles and vicinity, including the Hotel Utah and Annex at Salt Lake City, Utah, which cost about $1 200 000.

In 1918, he was made Engineer of Construction in the Concrete Ship Division of the Emergency Fleet Corporation and was instrumental in developing the concrete ship which, although an experiment, furnished one of the greatest engineering problems of the World War.

After the war, Mr. Bush was interested for a time chiefly in reinforced concrete. After a brief service with the Turner Concrete Steel Company, of Philadelphia, Pa., he became associated with Philip S. Tyre, Assoc. M. Am. Soc. C. E., and designed the structural work—steel and reinforced concrete—for many large buildings erected in Philadelphia during 1920, 1921, and 1922.

All his associates will remember Mr. Bush as a man of the finest character and a most capable structural engineer, the seasoned type with whom others delight to associate. A final tribute to his success and outstanding ability came in the form of a request from Edwin Clark, Assoc. M. Am. Soc. C. E., Chief of the Bureau of Building Inspection of Philadelphia, for him to reorganize the concrete inspection force of the Bureau after it had been impaired by illness of former employees. This, his last work, was a service well performed under the handicap of failing health.

He had patiently acquired, through study and actual practice, from 1902 until his death on October 21, 1923, a remarkable engineering equipment. His

* Memoir prepared by Philip S. Tyre, Assoc. M. Am. Soc. C. E.

well-worn technical library attests his mental journey through the highways and byways of engineering. In those ripe days before his death, he needed to consult few authorities in regard to his problems.

Mr. Bush was elected an Associate Member of the American Society of Civil Engineers on October 4, 1910, and a Member on March 2, 1915.

JAMES HENRY CUNNINGHAM, M. Am. Soc. C. E.*

Died July 28, 1923.

James Henry Cunningham was born at Edinburgh, Scotland, on September 11, 1847. Of an old Edinburgh family, he was the son of the late James Cunningham, and a grandson of Alexander Cunningham, a friend of Robert Burns.

James Henry Cunningham received his education at Edinburgh Academy, and after a year at Edinburgh University, he entered the office of Messrs. B. and E. Blyth, Consulting Engineers for the Caledonian Railway, as an articled pupil. After serving an apprenticeship of four years, he continued with the firm until 1875, having been employed in field work and in the preparation of plans and estimates for the Caledonian Railway Company. During this time, he was also engaged on plans and estimates for bridges and other work for the various railways constructed by the firm. During the last three years of his engagement, he served as Resident Engineer in full charge of the work on the Balerno Branch of the Caledonian Railway.

In October, 1875, Mr. Cunningham came to the United States, where he was employed until December, 1876, by the Kellogg Bridge Company. Subsequently, until February, 1878, he was with the Milwaukee Bridge Company. He then engaged in bridge construction as Engineer and Proprietor of the Milwaukee Bridge Works, Milwaukee, Wis., until 1881, when he returned to make his home in Scotland.

Having given up his engineering practice on his return to Edinburgh, Mr. Cunningham voluntarily devoted his time to the religious, educational, and social activities of the city.

In 1903, he was elected a member of the Edinburgh School Board, on which he served for eleven years, giving much time and work to the furtherance of educational progress. He also served for many years as a Director of the Edinburgh Royal Blind Asylum and also of the Deaf and Dumb Institution, in the welfare of which, he was greatly interested.

Mr. Cunningham was well known as an antiquary, and from 1891 to 1899, he served as Treasurer, and from 1899 to 1902, as Secretary, of the Society of Antiquaries of Scotland. He was prominently identified with a number of the investigations made by the Society, in which his engineering experience was of great assistance. He also took great pleasure in arranging excursions to view ancient Roman camps and historical buildings in Scotland.

* Memoir compiled from information on file at the Headquarters of the Society.

A few months before his death on July 28, 1923, Mr. Cunningham disposed of a number of interesting and important letters which had been written by Robert Burns to his grandfather, Alexander Cunningham, and which he had carefully preserved, to the Trustees of the Burns Cottage at Ayr, as relics of National importance.

He was married to Miss A. M. Pellatt, the daughter of Henry Allen Pellatt, who survives him.

He was a devout and sincere Churchman, and devoted himself unreservedly to the cause of the Scottish Episcopal Church. He served as a member of the Educational Board of the Church, taking especial interest in the Training College at Dalry House and the erection and furnishing of the College Chapel. It has been said of him that "his devotion was as real as it was simple and unaffected, and was equalled only by his large-hearted charity, his kindness, and his generosity."

Of his private life, it has been stated that "Mr. Cunningham was a fine example of the Scottish gentleman, devoid of pretence, with a certain reserve that came of sincerity, full of interest in all concerns that matter, hospitable and affectionate, with a wholesome detestation of humbug and unreality."

Mr. Cunningham was elected a Member of the American Society of Civil Engineers on August 6, 1879.

B. J. DALTON, M. Am. Soc. C. E.*

Died October 28, 1923.

B. J. Dalton, the son of George Washington and Adaline Harmon Dalton, was born in Franklin, Ky., on May 20, 1865. During his infancy, his parents moved to Illinois and shortly thereafter to Kansas. He was reared and educated under pioneer surroundings, which fact accounted in a measure for that rugged, stalwart, substantial, and true man into which he developed.

Mr. Dalton was educated in the public schools of Montgomery County and the High School at Independence, Kans. Subsequently, he attended the University of Kansas at Lawrence, from which he was graduated in 1890 with the degree of Bachelor of Civil Engineering.

Mr. Dalton began the practice of his profession in 1887, as Rodman on the construction of the Verdiquis Valley, Independence and Western Railroad. After his graduation, he resumed his engineering work as Transitman with the Union Pacific Railroad Company and was soon promoted to be Assistant Engineer. In February, 1891, he became Resident Engineer on the construction of the Texas, Louisiana, and Eastern Railroad. From 1893 to 1898, he was employed, successively, on preliminary surveys in Colorado, in private practice in Lawrence, Kans., and as Division Engineer on the construction of the Kansas City, Pittsburg, and Gulf Railroad. In 1898, he was made Chief Engineer of the Kansas, Oklahoma Central, and Southwestern Railroad, and, in 1900, he was appointed Assistant Chief Engineer of the St.

* Memoir prepared by John S. Worley, M. Am. Soc. C. E.

Louis and North Arkansas and the Arkansas and Choctaw Railroads. From 1903 to 1905, he served as City Engineer of Lawrence, and, in 1905 and 1906, as Chief Engineer of the Denver, Enid, and Gulf Railroad.

In September, 1906, Mr. Dalton was appointed to the Faculty of the Kansas State University as Associate Professor of Civil Engineering and Professor of Railway Engineering, where he remained until 1914.

When the law for the valuation of the public utilities of the State of Kansas was passed, Mr. Dalton's services were requested, and he spent fourteen months with the Public Utilities Commission.

The enactment of legislation by Congress having provided for placing values on all interstate railroads, the Interstate Commerce Commission organized its field forces by districts, and Mr. Dalton was made Assistant Engineer of the Western District, with headquarters at Kansas City, Mo. He remained in this position from 1914 to 1916, when he was appointed Chairman of the Valuation Committee of the Missouri, Kansas, and Texas Railroad, with offices at Parsons, Kans. He held this office until August, 1919, when failing health compelled him to relinquish it for that of Assistant Engineer, in which position, with lighter duties, he could still devote his time and remaining energies to the transportation problems of the country.

After an heroic struggle, during which he showed not only courage, but Christian fortitude and charity, Mr. Dalton passed to his reward on October 28, 1923.

On June 9, 1886, he was married to Louella Otwell and to that union there were born four children, three of whom, William B., Mrs. Marley R. Brown, and Mrs. George Henry Buecking, Jr., with Mrs. Dalton, survive him.

He was a man of sterling worth, his early surroundings having laid a groundwork of character that developed throughout his entire career. He was a hard and enthusiastic worker, simple in his tastes, but broad in his understanding of human nature and the problems of his profession. To his associates, he was always a friend.

Mr. Dalton was a member of the Sigma Xi and Tau Beta Pi Fraternities, and of the American Railway Engineering Association. He was a member of Acacia Lodge No. 9, Ancient Free and Accepted Masons, of Royal Arch Chapter No. 39, and of the Order of Eastern Star Chapter No. 49.

He had united with the Presbyterian Church in 1887 and was a devout member and active participant in church affairs, having served as Deacon and Elder until his health failed.

Mr. Dalton was elected a Member of the American Society of Civil Engineers on October 2, 1907.

BENJAMIN BRENTNALL LATHBURY, M. Am. Soc. C. E.*

Died November 10, 1922.

Benjamin Brentnall Lathbury, the son of Stephen and Margaret Conrad Lathbury, was born in Philadelphia, Pa., on March 9, 1870. He received his

* Memoir compiled from information on file at the Headquarters of the Society.

early education at the Lauderbach School and, later, entered the University of Pennsylvania, from which he was graduated in the Class of 1890, with the degree of B. S.

From June to September, 1890, Mr. Lathbury served as Draftsman and Computer in the U. S. Engineer Office, at Wilmington, Del., but resigned this position to return to the University of Pennsylvania, for a post-graduate course for the degree of C. E. While at the University, during the winter of 1891, he worked on maps, current observations, and soundings in the U. S. Engineer Office in Philadelphia, making copies for the Wilmington office.

Having received his C. E. degree, in May, 1891, Mr. Lathbury became U. S. Assistant Engineer in charge of a party on surveys of the Susquehanna River from Port Deposit, Md., to the head of Chesapeake Bay, for the Wilmington Office. This work included the triangulation of the surrounding country, measurements of base line, river soundings, current observations, platting of notes, etc.

From November, 1891, to January, 1893, he held the position of Assistant in the office of the Maintenance-of-Way Engineer, for the Pennsylvania Lines West of Pittsburgh, Cincinnati Division, at Cincinnati, Ohio, having charge of all field and office work on culverts, small bridges, retaining walls, and curves, incidental to maintenance of way.

From January to June, 1893, Mr. Lathbury served as Transitman and Levelman with the Construction Corps of the main line of the Pennsylvania Railroad, on change of line at various places at and between Glen Loch and Conewago, Pa., and the construction of new culverts and bridges, fills, cuts, track-laying, re-location of buildings, etc.

From June, 1893 to January, 1894, he was employed by Booth, Garrett, and Blair, as Inspector on engineering building materials and construction, and in laboratory work which consisted of the testing and inspecting of cements, steel, and brick.

From March, 1894, to January, 1897, Mr. Lathbury was a member of the firm of Lathbury and Anderson, Civil Engineers, with headquarters in Philadelphia. The work of the firm included the inspection of factory buildings and materials for building construction and the physical and chemical testing of materials. During this time (1895 to 1896), he also served as Chemical Engineer for the Alpha Portland Cement Company.

In January, 1897, the firm name was changed to Lathbury and Spackman, its members continuing in the same line of work which included the design and erection of cement plants for the Castalia Portland Cement Company, Castalia, Ohio; Alma Portland Cement Company, Wellston, Ohio; Clinton Cement Company, Pittsburgh, Pa.; Portland Cement Company of Utah, Salt Lake City, Utah; the Rotary Plant for the Aalborg Cement Works, Aalborg, Denmark; Rotary Plant for the American Cement Company, Egypt, Pa.; Lawrence Cement Company of Pennsylvania, Siegfried, Pa.; Wabash Portland Cement Company, Helmar, Ind.; American Works of the Alsen's Portland Cement Company, of Hamburg, Germany, at West Camp, N. Y.; Beaver Portland Cement Company, Marlbank, Ont., Canada; Portland Cement Plant of the Michigan Alkali Company, Wyandotte, Mich.; Aetna Portland Cement Com-

pany, Detroit, Mich.; a By-Product Plant for D. S. Warren and Company, Cumberland Mills, Me.; and the Almendares Portland Cement Company, Havana, Cuba. The firm also reconstructed the plants of the Auckland Portland Cement Company, of Auckland, and the Milburn Lime and Cement Company, of Dunedin, New Zealand. Mr. Lathbury laid out these plants, purchased all the machinery, designed all steel structures connected with the buildings and machinery, prepared all contracts and specifications, and superintended their erection.

In addition to this work, he acted as Consulting Engineer for a number of Eastern and Western cement plants in the matter of improving the general equipment, in order to reduce the cost of manufacture. The work of the firm also included acting as experts in examinations and reports on deposits suitable for the manufacture of Portland cement.

In the spring of 1904, Mr. Lathbury severed his connection with this firm and accepted the position of President and General Manager of the Alma Cement Company. He re-organized the Company, designed the entire plant, made all purchases, and superintended the erection until it was placed in operation. This work included laying out, purchasing and installing all machinery for the entire process, together with all steel buildings for the Wellston (Ohio) plant, the design and equipment of machine shops, storage buildings, office building, laboratories, side-racks for freight, coal, and limestone storage trestles, etc. He was responsible for the design and construction of all buildings, laying out, purchasing, and erecting all machinery appliances, and of the electric roads connected with the Company's mines at Oreton, Ohio. He opened and developed the entire property of 3 364 acres, with its deposits of coal, limestone, and shale.

When the United States entered the World War, Mr. Lathbury offered his services to the Government. On November 9, 1917, he was commissioned a Major in the Ordnance Department and was stationed at Washington, D. C., on construction work. When he was about to realize his highest ambition—active service abroad—the armistice was signed.

In 1918, he was promoted to the rank of Lieutenant-Colonel and, in 1919, to that of Colonel. During the winter of 1919, he was detailed on special work concerning the adjustment of claims in Canada and, in August, 1920, was sent to Haiti as head of an Army Commission and, again, in November of the same year, as a Special Envoy. On January 1, 1921, Mr. Lathbury resigned from the Army, and, on January 31, received the Distinguished Service Medal "* * * for exceptionally meritorious and distinguished services in the adjustment and settlement of many difficult and intricate claims problems", being especially commended for his "zeal, excellent judgment, and high comprehension of the principles involved."

He died suddenly of angina pectoris on November 10, 1922, being survived by his widow, Mrs. Bertha Nesbitt Lathbury, a daughter, and a son.

He was a member of the Union League and University Clubs of Philadelphia, the Sons of the American Revolution, and the Sons of Sweden.

Mr. Lathbury was elected an Associate Member of the American Society of Civil Engineers on March 4, 1897, and a Member on June 6, 1905.

ROWAN AYRES, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 13, 1912.

Rowan Ayres, the son of S. C. Ayres, M. D., was born at Cincinnati, Ohio, on August 27, 1876. He received his technical education at Yale University, having been graduated from the Sheffield Scientific School with the Class of 1898.

After his graduation, in July, 1898, Mr. Ayres entered the employ of the Cincinnati Telephone Company, remaining with that Company until March, 1899. He then went to Mexico as Assayer for the San Carlos Copper Company, and, except for brief visits to the United States, remained in that country until his death.

After leaving the employ of the San Carlos Copper Company in July, 1899, until October, 1908, Mr. Ayres was engaged with various mining companies, as Assayer and Engineer on construction work, including the Montezuma Lead Company, the Penoles Mining Company, the Animas Mining and Milling Company, etc. He was also employed on the location and construction of several branches of the Mexican International Railroad in various capacities, including that of Engineer in charge of construction.

After a year of private practice, with an office at Durango, Mr. Ayres was again engaged in mining operations and, later, on railroad location and construction for the Nazas Valley and Pacific Railway Company until 1911. From July, 1911, to February, 1912, he was in charge of the construction of a lumber road for the Cia. Maderera de la Sierra Durango, Parker and Carroll, Contractors.

In February, 1912, he was appointed Superintendent of Construction for the Cia. Constructora de Ferrocarriles on the construction of a line for the National Railways of Mexico from Penjamo, Guanajuato to Ajuno, Michoacan. While engaged in this work, he was brutally murdered by Mexicans on August 13, 1912, at Patzcuaro, Michoacan. His body was recovered and brought to his home in Cincinnati for burial.

As an engineer, Mr. Ayres was successful both in the field of mining engineering and that of railroad location and construction. He was a man of fine character and sterling honesty and a credit to his American citizenship in a foreign land. He was unmarried and was survived by his father and mother, by two brothers, W. McL. Ayres, M. D., Cincinnati, Ohio, and Robert W. Ayres, U. S. Forest Service, California, and two sisters, Mrs. A. Hamilton Rowan, Bletchingly, Surrey, England, and Mrs. Pelham H. Blossom, Cleveland, Ohio.

Mr. Ayres was elected an Associate Member of the American Society of Civil Engineers on July 9, 1912.

* Memoir prepared by Hiram Miller, Assoc. M. Am. Soc. C. E.

ANDREW FRANCIS ROSS, Assoc. M. Am. Soc. C. E.*

Died May 8, 1921.

Andrew Francis Ross was born at East Dedham, Mass., on June 16, 1863. His father was of Scotch and his mother of Irish descent. He obtained his early education in the public schools of Dedham and, later, studied chemistry and engineering under a private tutor. Mr. Ross was largely self-educated, having been a keen observer and a persistent reader and student of technical literature. His work as an engineer was varied in character, embracing mining, railroad engineering of many kinds, difficult tunnel construction, subways, earth and concrete dams, stone arch bridges, irrigation works, and general construction.

On leaving school in 1881, he went West and engaged in mining work in Nevada, and, in 1886, became Assayer for the Chrysty Mining Company, at Silver Reef, Utah. In 1887, he was employed by the Pioche Consolidated Mining Company of Pioche, Nev., and, in 1889, was made Superintendent of the Spring Mine at Pioche.

In 1890, Mr. Ross returned East and, until 1896, was engaged in railroad work, serving, successively, as Transitman on construction for the New York, New Haven and Hartford Railroad Company; Assistant Engineer for the Plymouth and Middleboro Railroad Company; and Transitman in charge of party on location, Engineer of Maintenance of Way on the Cape Cod Division, and Assistant Engineer on plans and estimates for the elimination of grade crossings at Brockton, Mass., for the New York, New Haven and Hartford Railroad Company. In 1896 and 1897, he was Superintendent of the Hecla and Silver Mountain Mining Company, at Empire, Colo.

In 1898 and 1899, Mr. Ross was Superintendent for Holbrook, Cabot and Rollins, Contractors, in charge of the construction of bridges and retaining walls on the Fitchburg Railroad, in Massachusetts, the Bellows Falls stone arch bridge, and a masonry arch bridge over the Tomhannock River at Schaticoke, N. Y. From 1900 to 1902, he supervised, for the same firm, the erection and operation of mechanical equipment on Section 3 of the New York Rapid Transit Subway. While on this work, he invented a number of ingenious labor-saving devices, which, with his characteristic greater regard for the efficient performance of work than his own personal profit, he neglected to patent. Many of these devices were patented afterward by others and are now being manufactured.

Mr. Ross moved to Arvada, Colo., in 1903, and for two years was engaged in farming. The fascination of construction work, however, was too great for him to be long contented with the quiet and isolated life of a farmer, and, in 1905, he entered the United States Reclamation Service, as Outside Superintendent of Road Construction and Plant Erection at the Gunnison Tunnel, on the Uncompahgre Irrigation Project, near Montrose, Colo. Except for a few short periods, he remained in the service of the Government until his death.

* Memoir prepared by Charles P. Williams, M. Am. Soc. C. E.

In 1907, he became Superintendent in charge of the construction of the Gunnison Tunnel and was employed later on the construction of other important irrigation structures on the North Platte, Belle Fourche, Sun River, Shoshone, and Strawberry Valley Irrigation Projects. While engaged on the repair of the concrete lining of the Strawberry Valley Tunnel, he contracted a severe cold, which, developing into pneumonia, resulted in his death on May 8, 1921.

In 1888, Mr. Ross was married to Miss Catherine McDonough, at Pioche, Nev., by whom he had two children, a son, Charles, who was killed at the battle of Chateau Thierry, and a daughter, Mrs. J. R. Cheney, of Durango, Colo., who, with his widow, survives him.

Mr. Ross was elected an Associate Member of the American Society of Civil Engineers on June 11, 1917.